THE UNIVERSITY OF ALBERTA

STRUCTURAL COMPARISON OF A SAND-ASPHALT WITH A SOIL-CEMENT

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE

STUDIES IN PARTIAL FULFILMENT OF THE

REQUIREMENTS FOR THE DEGREE OF

MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

BY

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EDMONTON, ALBERTA

APRIL, 1962

ACKNOWLEDGEMENTS

The author wishes to extend his appreciation to:

Mr. K. O. Anderson, Associate Professor of Civil Engineering for his constructive criticism and generous lending of publications from his personal library.

The Alberta Research Council, Highways Division, under the co-operative Highway Research Program for supplying the materials and laboratory assistance for the investigation.

Mr. B. G. Hutchinson for items (7) and (14) of the Bibliography.

Mr. N. Slipchuck for his assistance in the laborious task of sample preparation.

ABSTRACT

The strength characteristics of a sand stabilized with a MC3 cutback asphalt were compared to the sand stabilized with Portland cement. For evaluation of the strengths of the two types of stabilization, triaxial compression testing was utilized. The soil-cement was found to be structurally superior to that of the sand-asphalt using triaxial testing but to a lesser degree than revealed by the unconfined compression test.

The testing program comprised 7 day cured and 14 day immersed series of sand-asphalt, cured untreated sand, 7 and 28 day cured soil-cement and 14 days immersed soil-cement series. Only 8 per cent cement was employed in the soil-cement but 3, 5, and 7 per cent mixes of MC3 sand-asphalt were utilized. The 3 per cent MC3 was found to be the optimum mix for the sand-asphalt.

The asphalt prevents the destruction of the sand specimen upon immersion but lowers the strength of cured specimens. Although the asphalt retards the entrance of water into the asphalt stabilized specimen a considerable quantity of water is absorbed. Due to the low permeability of the soil-cement, water absorption is small and thus there is no strength decrease but rather an increase due to the beneficial hydration conditions.

Using the design methods of Kansas, Texas and Burmister's theory of two-layer systems, it was shown that the soil-cement possesses much higher strength than does the sand-asphalt.

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CHAPTER I

PURPOSE AND PREVIEW OF INVESTIGATION

As gravel sources become depleted or become too far distant for economic use, it will be necessary to replace gravel as a base course material, with materials such as fine sands and silts stabilized with cementing or waterproofing admixtures.

This thesis is one of a continuing series of investigations on stabilization of highway materials sponsored by the co-operative Highway Research Program in Alberta. Recent investigations have been carried out using lime-pozzolan for stabilizing a plastic clay (1)* and cement stabilizing of sand (2). One objective of the Highway Research Program in Alberta is to compare the methods of stabilizing inferior materials. This dissertation will only attempt to compare cement stabilization with asphalt stabilization.

Since 1953, 118.86 miles of soil-cement base course have been constructed in Alberta by the Department of Highways. This total mileage represents approximately 2,881,000 square yards of soil-cement base course (3). As non-plastic soils are still in plentiful supply, only sands were employed in the soil-cement base courses. Consequently, asphalt stabilization of a sand will be attempted.

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*Numbers in brackets refer to references in the Bibliography.

In the United States 6700 miles of soil-asphalt stabilized base courses and sub-bases have been laid with varying degrees of success (4). A considerable amount of asphalt stabilization has also been performed in the province of Manitoba.

For this investigation a sand was selected which had been utilized in a soil-cement base course on Highway 16 near Stony Plain, Alberta. Since this sand was stabilized successfully with cement, it is desired to determine its suitability when stabilized with asphalt.

If a road-mix type construction procedure were employed, it would be necessary to use an asphalt which could be mixed with the sand at normal temperatures. Therefore, a MC3 cutback asphalt was chosen for this project. The emulsions and asphalt cements are frequently used if a single-pass stabilizing procedure is available.

From the available literature, it may be concluded that soil-asphalt stabilization has been used successfully in many regions. Soil-cement has proven very satisfactory in Alberta and has attained a high degree of success both economically and structurally (3). Consequently, the purpose of this investigation was two-fold:

(1) Ascertain the significant physical properties and characteristics of a MC3 stabilized sand.

(2) Compare the merits and shortcomings of a sand stabilized with cement as compared to the same sand stabilized with a MC3 asphalt.

Although there are different criteria for comparing materials, the most common is that using strength relationships. The methods for

strength comparison are many and varied (5) (6). The method selected for this investigation was the standard triaxial test with positive lateral pressure. The lateral pressure which is maintained in the triaxial test tends to simulate the confining pressure under which a base course is subjected.

The mechanics of sand-asphalt* as understood at present is for the asphalt to add cohesion to cohesionless sands and/or waterproof the compacted mix (5). Consequently, it was the purpose of this investigation to determine the ability of a MC3 asphalt to maintain the strength of a cohesionless sand under immersed conditions. The effect of asphalt on cured strength was also to be investigated. Curing with reference to asphalt stabilization refers to the driving off of volatiles and water after compaction. Of course, the cured condition is the ideal condition for an asphalt stabilized material.

To obtain a comparison of soil-cement stabilization with the sand-asphalt stabilization three series of soil-cement specimens were performed including seven day cure, twenty-eight day cure and fourteen days immersion.

The drawing of a best-fit line to obtain the Mohr rupture line for a given number of triaxial tests is dependent on personal judgment and experience. Therefore, it was decided to use an analytic solution

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*Asphalt stabilization is generally referred to as "soil-asphalt" for any gradation of material but frequently when sand is utilized the term "sand-asphalt" is used. Therefore, "sand-asphalt" will be used throughout this thesis when referring to sands.

for Mohr's envelope. The method was introduced by Balmer (7) and is based on the least squares procedure for fitting a line to experimental data. Judgment must be exercised in deciding upon the nature of the function which best describes the experimental data. For analysis in this investigation a linear envelope is assumed. One advantage of this analytic fitting method is that it is possible to derive confidence limits as a measure of the reliability of the fitted line. Consequently, Mohr rupture circles will not be included in this thesis but rather the results of friction angle, cohesion and confidence limits will be shown in tabular form.

For analysis and comparison of the soil-cement and sand-asphalt admixtures the design methods used by the states of Texas and Kansas will be applied to obtain thicknesses for the materials. A method formulated by Hank and Scrivner and based on Burmister's theory of stresses in a two-layer system will also be utilized in an analysis. Of course, the most straightforward approach to comparison is a collation of friction angles, cohesion and stress-strain curves.

An annotated asphalt bibliography is contained as part of this dissertation and contains a summary of references available since 1935.

CHAPTER II

REVIEW OF LITERATURE

Stabilization Methods

Asphalt is one of many types of stabilizing agents. Depending on the mechanism of stabilization these stabilizers may be subdivided into three general groups (5).

The first group includes materials which cement the soil particles. These materials, capable of reactions within themselves or with certain soil constituents in the presence of water, produce strong interparticle bonds which can support high intensity loads. Typical representatives of this group are cements, limes, and more recently acidic phosphorus compounds. The soil stabilized with these materials may possess high initial strengths but because of the nature of the bonds formed, lack desirable durability characteristics when exposed to such environmental conditions as drying-wetting or freezing-thawing.

A second group of stabilizers could be termed the soil modifiers or conditioners. Lime, calcium, or sodium chlorides, and a number of surface active materials are typical of this group. Depending on the character of the soil, improvements in mixing, drying, compaction, strength and other wet soil properties can be realized with small quantities of these modifiers. But just as in the case of the cementing agents, soil masses treated with these modifiers are highly susceptible to climatic changes.

The third group embraces the waterproofing agents. Asphalts, certain resinous materils, and coal tars are representative of this group. These stabilizers coat individual soil particles or their agglomerates, and thereby prevent or hinder the penetration of water into the stabilized soil layer. The role of the asphalt film in stabilizing a soil mass depends to a great extent on the properties of the soil. In the case of non-cohesive soils (sands and silts) the asphalt film serves a double purpose. First, it waterproofs the soil mass. Second, it binds the soil particles together giving the material cohesion.

With the advances in soil mechanics during the last several decades, long strides have been made toward a better understanding of soil-asphalt systems. Knowledge of the basic properties of such systems remains relatively sparse. Little is known, for instance, about determining the composition of the mixture or thicknesses of the stabilized compacted layer to be used.

Many secondary additives have been tried in soil stabilization with asphalts. Their principal purpose is to condition the soil and improve asphalt adhesion to the mineral surfaces.

Mechanics of Asphalt Stabilization

The most important objective of soil stabilization is the elimination of the dependency of soil engineering properties on moisture content. The most obvious way in which the water sensitivity of soils can be eliminated is to coat the individual soil particles with a

film of a stable, water-insoluble resinous substance which will prevent water from reaching the particle surfaces, and/or prevent water from migrating by capillarity into a consolidated porous soil mass. Use of a resinous material provides further advantages in that it causes the individual particles to adhere to one another, and thus contributes to the shear strength of the compacted mass.

Undoubtedly, the most attractive substances in Alberta (from the standpoint of availability and cost) falling into the above category are the petroleum derived asphalts.

Asphalt as a soil stabilizer is, however, limited by certain physical properties both of the asphalt and of the soil. The three most important factors limiting its effectiveness as a stabilizer are believed to be: (1) the difficulty of distributing asphalt uniformly throughout a material such as clay or fine sand (2) the inability of asphalt to adhere to wet soil particles and (3) the sensitivity of the asphalt-soil bond to destruction by water.

To facilitate distribution of asphalt through soils, and thus to improve its stabilizing action by minimizing the first factor cited above, two basic approaches have been followed in practice. One is to increase the fluidity of the asphalt at useful temperatures by diluting it with solvent; the other is to emulsify the asphalt in water. Both processes yield mobile liquids which can be admixed with soils more effectively than asphalt alone. In addition to the two methods mentioned above, a relatively new method has been developed for mixing asphalt

with soil, in which the asphalt cement is in the form of a foam (8). In this investigation an MC3 asphalt is utilized which has kerosene as the cutbacking agent.

Mechanism of Cutback Stabilization

Stabilization of soils with asphalt cutback is normally carried out by: (1) blending the soil at its optimum water content with the cutback (2) compacting the mixture and (3) curing the mixture by allowing water and cutback solvent to evaporate over a period of time. Inasmuch as the blending process involves mixing a water-wet soil with a water-immiscible fluid which does not adhere to the soil-particles, the mixing process can be envisioned as one of subdivision of the cutback fluid into small droplets which remain insulated from the soil particles by films of water. On compaction the mass is probably composed of a skeletal structure of soil with water and cutback-globules trapped in the voids. As water evaporates from the mixture, it becomes possible for the asphalt cutback to wet out and adhere to a larger fraction of the soil particle surfaces. As this wetting-out process proceeds, the asphalt would distribute itself more uniformly provided it had the required fluidity to do so. This fluidity depends primarily upon the rate of evaporation of the cutback solvent; rapid curing cutback would be expected to lose solvent and thus increase in viscosity, with the result that little improvement in asphalt distribution might result during curing. With slow-curing cutbacks, the slower the rate of solvent evaporation, the slower the rate of development of cohesive strength and thus the slower the rate of strength-buildup on curing.

On immersion in water after curing, water is undoubtedly imbibed very rapidly by those regions of the mixture which have not been adequately protected by asphalt; this should result in an initial rapid loss in strength. In view of the water sensitivity of the asphalt-soil bond, water should gradually find its way into the asphalt-soil interface and destroy this bond; the rate of detachment should be dependent upon the firmness and extensiveness of the bond developed, and the fluidity of the asphalt. This would indicate that there should be, on continued water immersion, a further gradual reduction in strength (and increase in water absorption) the rate of strength loss being greater for incompletely cured mixtures or those prepared with less viscous asphalts.

Drying and Curing of Soil-Asphalt

It has been recognized in soil-asphalt stabilization that the drying and curing of the mixture is very important in order to obtain the most stable material. Some authorities state that after mixing cutback asphalt with a soil containing the required water that before compaction the mix should be dried down to a low moisture content and then compacted.

Results presented by Herrin (9) indicate that for a silty sand, stabilized with MC3 that the mix needed to be dried out before compaction to provide high initial stability. Although it is stated that drying is needed before the mixture is compacted, too dry a mixture is not desirable. The drier the mixture is at the time of compaction, the greater the air-void content and consequently, the mixture is more susceptible to soaking up water. Herrin also concluded that soil stabilized with cutback asphalt cannot be compacted to a density requirement, for high strength is not related to high unit weight in soil-asphalt mixtures. Thus, any control of

the stability of these mixtures by a density requirement should be accompanied by a limitation in the amount of water and hydrocarbon volatiles in the mixture.

A paper by Katti, Davidson and Sheeler (10) reports that compaction of a soil-cutback asphalt-water system immediately following mixing produces a more stable product than a procedure in which a drying back period is included between mixing and compaction. It was also concluded that the percentage of mixing water required to produce maximum strength, maximum standard Proctor density, minimum total moisture content after seven days immersion, and minimum expansion in compacted specimens is different for each property mentioned. However, the range of water contents is only very small. Katti's research included five soils with different asphalt cutbacks and his results appear to contradict Herrin's work stating that a asphalt-soil mix cannot be compacted to a density requirement. According to Katti's work maximum strength is coincident with maximum density.

In this investigation the procedure of mixing, compacting without drying back and then curing was employed.

CHAPTER III

CLASSIFICATION AND ANALYSIS OF MATERIALS

Sand

Only one type of soil was used in this investigation, that being a sand of poor grading. The source of the sand was McGinn Pit North (SE 1/4-5-53-R1-W5) near Stony Plain, Alberta. A visual examination of the native sand plus grading tests showed that the material corresponded to the SP soil group of the Casagrande Airfield Classification System (11) and a A-3 type soil according to the AASHO soil classification (12).

After being received from the field the sand was thoroughly mixed and air-dried. For storage and future use the material was placed in canvas bags in portions of seventy pounds.

The uniformity coefficient of 2.58 indicates a material which contains a large portion of particles the same size.

Table 1 shows the results of the average of three sieve analysis and other classification data for this sand. The results of a hydrometer analysis is shown in Appendix B.

Cement

For preparation of soil-cement specimens, a Type 1 normal Portland cement was utilized which was supplied by Inland Cement Company Limited.

The cement was stored in sealed polyethylene bags to prevent

moisture absorption and hydration.

The basic classification items are listed in Table 2.

MC3 Asphalt Cutback

The cutback asphalt was supplied by Husky Oil and Refining Company Limited of Lloydminster, Alberta. The analysis of the MC3 was performed by the supplier and is shown in Table 3.

The asphalt after being received from the supplier was placed in a large thermostatically controlled barrel at a temperature of 175° F.

Although the supplier's analysis shows 79 per cent residue, it was found that by obtaining a MC3 sample and placing it in an oven at 105° C. that the residue was 75 per cent. This may be due to variation in sampling. Consequently, the residue percentages referred to will be based on 75 per cent asphalt residue.

TABLE 1

Classification and Analysis of Sand

AASHO Soil Type	A - 3
Specific Gravity	2.67
Standard Proctor Density	104.0 lbs/ft^3
Optimum Moisture Content	15.0 %
Uniformity Coefficient	2.58

Sieve Analysis of Sand

U.S. Sieve No.	% Material Passing
20	100.0
40	99.3
60	76.4
100	24.7
200	8.1

TABLE 2

Analysis of Cement *

Type of Cement	Type 1 Normal Portland
A.S.T.M. C109 Cube Strength	3400 lbs/in^2
Specific Gravity	3.10

*Cement analysis performed by Alberta Research Council, Highways Division

TABLE 3

Analysis of MC3 Cutback

Specific Gravity at 60° F. 0.9792

API Gravity at 60° F. 13.0

Flash Point (T. O. C.) . . . . . 150+ °F.

Water % . . . . . Nil

Saybolt Furol Viscosity at 140° F. . . . 392 sec.

Distillation . . . .

	% of Total Over	% of Distillate Total Over
437° F.	0	0
500° F.	2	9.3
600° F.	14.5	69.1
680° F.	21.0	100.0

Residue to 680° F. volume by difference . . . 79.0%

Residue Penetration at 77° F. (100 gms., 5 sec.). 155

Soluble in Carbon Tetrachloride 99.8%

Ductility at 77° F. (5 cm. per min.) . . . 100 cm.

CHAPTER IV

TESTING PROGRAM

Outline of Testing Program

Prior to the triaxial testing program it was necessary to determine the optimum moisture content for the asphalt contents to be investigated. The criteria for optimum moisture was maximum dry density based on the dry weight of the total mix. These specimens were broken in unconfined compression to determine whether a sand-asphalt material could be compacted to a density requirement for maximum strength. The MC3 contents selected were 3, 5, and 7 per cent with residue amounts of 2.25, 3.75, and 5.25 per cent, respectively.

The specimen size selected for the triaxial program was 2.8 inches in diameter by 6.5 inches long. These dimensions gave a height to diameter ratio of 2.32 which is between the ratio of two to three recommended by many authorities for cohesionless materials (13).

The sand-asphalt testing series was divided into two groups: (1) specimens cured for seven days at 100° F. (2) specimens cured seven days at 100° F. and immersed for fourteen days. For each asphalt content and curing condition, a total of twelve specimens were tested, which comprised nine triaxial and three unconfined tests. Thus, three samples were tested at each particular lateral pressure.

The sand-asphalt specimens were cured for seven days as

it was indicated from preliminary testing that the volatile content reached a minimum and thus the cured strength reached a maximum.

One series of tests was performed on the pure sand compacted at optimum moisture content and cured at 100° F. In addition, three pure sand samples were immersed after the seven day curing period.

The soil-cement specimens were compacted at fifteen per cent water with eight per cent cement. This design mix was that which was used in the soil-cement base course on Highway 16 near Stony Plain, Alberta.*

To obtain a comparison of soil-cement stabilization with the sand-asphalt stabilization, a seven day cured series of soil-cement specimens was prepared to obtain the Mohr rupture envelope. Although design methods for soil-cement are based on the seven day strength, the twenty-eight day strength is considered to be available, although the strength increases over longer periods of time. Thus, it was decided to determine the increase in strength from seven day cure to twenty-eight day cure under triaxial conditions. The ideal curing conditions for soil-cement occur when water does not evaporate from the mix and is thus available for hydration of the cement. Therefore, in this investigation, curing was obtained by sealing the specimens in plastic bags.

As indicated by many authorities there is a considerable drop in strength of asphalt stabilized materials upon immersion, conseq-

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*Design received verbally from B. Hutchinson of the Alberta Research Council, Highways Division.

uently it was desired to determine the effects of immersion upon soil-cement.

Determination of Optimum Volatile Content for Triaxial Specimens

The optimum* volatile content for the individual asphalt mixes is comprised of both the water and cutback solvent. In MC3, the asphalt cutback is kerosene. Thus, any reference to optimum volatile conditions in relationship to sand-cutback admixtures will include the cutback agent as well as the water.

Before determination of the optimum volatile content of the individual asphalt mixes it was necessary to simulate Standard Proctor density in the 2.8 inch diameter mold. The optimum moisture content of the pure sand was determined in a Standard Proctor mold using the Standard Proctor field apparatus. Then by trial and error the number of blows of a 10.3 pound hammer with a 2.8 inch diameter compacting foot, dropping 18 inches, required to duplicate Standard Proctor density was determined for the pure sand in the 2.8 inch diameter mold. It was necessary to apply 10 blows on each of five layers to duplicate Standard Proctor density. This blow count resulted in compactive energy of 34,200 ft-lbs per cubic foot compared to 12,400 ft-lbs per cubic foot applied in Standard Proctor compaction. The excess energy utilized is believed to be required to overcome the side friction of the mold. This is discussed in the last section of this chapter.

To reduce the effects of density gradient, the number of blows was varied per layer with 8, 9, 10, 11, and 12 blows on five layers

* Optimum volatile content is that volatile content at which maximum density is obtained.

with increasing blows per layer upwards.

Tests performed by the Alberta Research Council* to determine the density variation by this method of compaction showed a definite decrease in density from the bottom of the sample to the top. The average variation in density from the top to the bottom was found to be four pounds per cubic foot. Tests were only performed on the soil-cement specimens. This density variation was not discovered until after testing was complete, although a preliminary determination of density variation in the sand-asphalt specimens was found to be less than two pounds per cubic foot.

In determination of the optimum moisture content for the 3, 5, and 7 per cent mixes, five moisture contents were selected which would give a complete moisture-density curve. After curing in an oven at 100° F. for seven days the specimens were broken in unconfined compression, thus obtaining a total volatile-strength relationship. After testing, the broken samples were dried in an oven at 105° C., thus giving the total volatile content at compaction including both cutback and water.

For obtaining moisture-density curves three specimens were tested at each molding volatile content. Finally, the optimum volatile contents determined were used to prepare all the samples for the sand-asphalt triaxial program.

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*Data obtained from W. Sadoway of The Alberta Research Council, Highways Division

Preparation of Specimens

(1) Sand-Asphalt Specimens

The MC3 cutback asphalt was heated constantly to 175° F. in a sealed barrel. Air-dried sand was weighed out in portions of 1400 grams and the required percentage of water was introduced and mixing was continued for two minutes with a large spoon. The required weight of asphalt, (based on the dry weight of the sand) was then added and mixing was obtained with a spoon for one minute followed by hand mixing for three minutes. The sand-asphalt mixture was then compacted in the 2.8 inch diameter mold. Compaction of specimens was carried out on a concrete block and compaction was maintained on one face due to the use of a solid base (Photograph 1). Each layer was scarified with a screwdriver to eliminate the detrimental effects of smooth compaction planes.

Extruding of the specimens was accomplished by using a Tinius Olsen 30, 000 kilogram compression machine. Initially, the 2.5 inch long base was forced into the mold flush with the bottom, after which the specimen was trimmed flush with the top. Final extrusion resulted in the 2.8 inch diameter by 6.5 inch long specimen.

The specimens were then placed in an oven at 100° F. for seven days. The cured series was tested immediately after the curing period. The immersed series, following curing, was placed in a large tank at room temperature for fourteen days with one inch of water covering the sample. Photograph 2 shows samples after

twenty days immersion except the pure sand sample which disintegrated after five minutes immersion.

(2) Soil-Cement Specimens

Air-dry sand was weighed out in portions of 1400 grams and eight per cent cement was added (based on the dry weight of the sand). The ingredients were then mixed dry for one minute by hand and then fifteen per cent water (based on dry weight of sand and cement) was then introduced. Mixing was again accomplished by hand but continued for three minutes.

The compaction technique for the soil-cement was similar to that of the sand-asphalt. Trimming of the soil-cement samples was also similar to that of the sand-asphalt samples except that extruding was not difficult and hence was performed manually.

After extrusion the samples were then covered with a plastic sheet and placed in the concrete moist room, at 100 per cent relative humidity. Because the samples could not be handled after extrusion without damage it was necessary to place them in the moist room for twenty-four hours of curing. There was no weight change during this curing period. After the initial curing the specimens were placed in polyethylene bags which were then sealed with wax. The final curing periods of six days or twenty-seven days was obtained in the sealed bags which were placed in a moist room at 70 per cent relative humidity. Immersed specimens were placed in the same tank as the sand-asphalt samples.

Apparatus and Procedure for Triaxial Tests

The list of apparatus used in conjunction with the triaxial testing programme follows:

(1) A Tinius Olsen 30, 000 kilogram capacity hydraulic compression testing machine was used for extruding the samples from the compaction mold (Photograph 3).

(2) Two 2.8 inch diameter triaxial compression cells were used to test the specimens. Employing two cells allowed setting up and preparing a sample while compression of another specimen was underway (Photograph 4).

(3) A Farnell loading machine was utilized for applying a uniform strain rate of 0.01 inches per minute (Photograph 5).

(4) For application of the air over water lateral pressure, the screw control of a pore-pressure measuring apparatus was utilized but with the pore-pressure gage disengaged (Photograph 5). Although the duration of a test seldomexceeded thirty minutes, it was necessary to adjust the pressure numerous times in order to maintain the cell pressure constant.

(5) All samples and ingredients were weighed on a 4000 gram capacity scale with one gram as the smallest division.

(6) The load application to the specimen was measured by means of proving rings. For the sand-asphalt specimens a 1500 pound capacity ring, No. 783 was used and for the soil-cement and pure sand specimens a 4000 pound capacity ring,

No. 867, was used. The calibration curves supplied by the manufacturer were checked and found to be correct.

Before testing the immersed sand-asphalt specimens, diameters and lengths were recorded. The weight of all specimens was obtained to the nearest gram before testing.

The specimen was placed on the base of the triaxial cell and a rubber membrane was attached to the base and loading head of the cell by means of O-rings and airplane rubber. Then the cell was assembled and filled with water. The top portion of the cell, around the loading piston was filled with a non-foaming oil to provide a air seal as well as prevent damage to the piston. The piston was then seated by hand as water was released from the bottom of the cell.

The cell was now placed on the loading table of the Farnell automatic tester and the loading yoke was brought into contact with the piston. After connecting the pressure lines the cell pressure was applied using the screw control of the pore-pressure apparatus. The recording dial of the proving ring was then zeroed so that the measured load would be the deviator load. The strain rate selected was 0.01 inches per minute as it was indicated from preliminary testing that a slower or faster rate had negligible effects on the strength properties.

Leaks in the rubber membranes were detected immediately by a significant drop in the cell pressure. Of course, when leaks developed it was necessary to dissemble the cell and reset with a

new membrane.

Finally, loading was commenced with proving ring readings being taken at strain reading increments of 0.1 per cent or 0.05 per cent strain.

At failure, that is when the deviator stress dropped, loading was discontinued, type of failure noted and samples taken from the cell and placed in an oven at 105° C.

The lateral pressures used were 30, 20, 10 and 0 pounds per square inch with three specimens at each lateral pressure. The soil-cement specimens were tested similarily to the sand-asphalt except that lateral pressures of 80, 50, 20, and 0 pounds per square inch were employed.

For the three series of soil-cement specimens, diameters and lengths were obtained for all specimens before testing.

Discussion of Procedure and Apparatus

The 2.8 inch diameter forming mold used for making the triaxial specimens was manufactured especially for this investigation from a thick-walled steel pipe. In the initial stages of the sample preparation much difficulty was encountered in simulating standard Proctor density in this mold. In the beginning the inside walls of the mold were quite rough although the machining had been skillfully performed. A large number of pilot samples were prepared and indications were that the mold was becoming smoother with use, thus

reducing the effects of wall friction. In addition to the fact that the smoothness of the mold could be seen to increase, the increase in density for the same compaction procedures also indicated that the mold was becoming polished. By the time triaxial specimens were fabricated, there was no indication that the walls of the mold would change in characteristics.

The first outline and proposal for this program, indicated that a double-plunger type mold with dynamic compaction and layers would be the most suitable method, thus eliminating the density gradient that is common in static compaction procedures. The double plunger method was tried but when compacting dry samples the plunger base would not move into the mold and thus not give compaction from the bottom. Conversely, when wet samples were compacted the base moved a considerable distance into the mold, causing the wet samples to have a higher density than would correspondingly be obtained by the compaction type obtained in the drier samples.

Finally, it was decided to use a solid base which prevented compaction from the bottom. This would no doubt result in increased density of the lower layers compared to the top layers. In order to partially compensate for this density variation, the number of blows per layer was increased from bottom to top. Therefore, instead of using ten blows per layer, which was found to simulate Standard Proctor density, blows of 8, 9, 10, 11, and 12 were arbitarily employed. The five layer sample was used as it was felt that a specimen length of 6.5 inches should be compacted in more lifts than the three used in the

Standard Proctor mold and thus reduce the density gradient.

The curing period of seven days for the sand-asphalt specimens was selected as it was indicated from preliminary testing that the strength levelled off after five days. Until a curing duration of five days was reached it was found that a time difference of six hours in testing could greatly affect the results of specimens which were considered to be identical. Thus by choosing a period of seven days a delay in testing of one day would not change the moisture conditions of the sample and thus not significantly affect the strength. Generally, it was found that the volatile content of the specimens after five or seven days curing was between 0.5 and 1.0 per cent.

Although all the sand-asphalt specimens were cured to the same final condition, it must be emphasized that this curing procedure is extreme and would not easily be obtained in the field. The curing temperature of 100° F. was arbitarily selected.

The immersion period of fourteen days was arbitarily selected as being a test that would show the effectiveness of asphalt as a waterproofing agent. After fourteen days immersion, water continued to be absorbed.

For determination of the volume of the cured samples, it was assumed that the change in volume of the cured specimens was negligible through desiccation. Consequently, the volume of the cured samples was assumed constant at 0.0226 cubic feet, which was the volume of the forming mold. All immersed specimens were measured and found to increase very slightly in volume due to the immersion

period of fourteen days. The method of volume determination was by utilization of a pair of calipers and it was very difficult to obtain consistent readings due to the loose sand on the surface of each specimen. In addition, volume determination by water displacement did not tend to indicate any significant change in volume. Thus all samples were assumed to have constant volume.

The strain rate of 0.01 inches per minute which was selected for the triaxial testing was chosen mainly for convenience. Although preliminary testing showed that there was no significant reduction in strength with slower rates this cannot be concluded due to the limited pilot tests. The slowest rate used, 0.005 inches per minute, resulted in a testing duration of two and one-half hours. This length of test for one hundred triaxial specimens and seventy-five unconfined specimens would have been prohibitive for the time available. Thus the average length of test was one-half hour using a strain rate of 0.01 inches per minute.

The lateral pressures of 30, 20, 10 and 0 pounds per square inch which were selected for the sand-asphalt specimens gave a convenient spread of Mohr's circles for determination of the rupture envelope. When testing the soil-cement specimens it was necessary to increase the lateral pressures so that a greater spread of Mohr's circles could be obtained. Initially, it was planned to determine the Mohr envelope by the conventional method of drawing a line tangent to the Mohr circles. If the circles are spread apart, the envelope is defined with much greater accuracy.

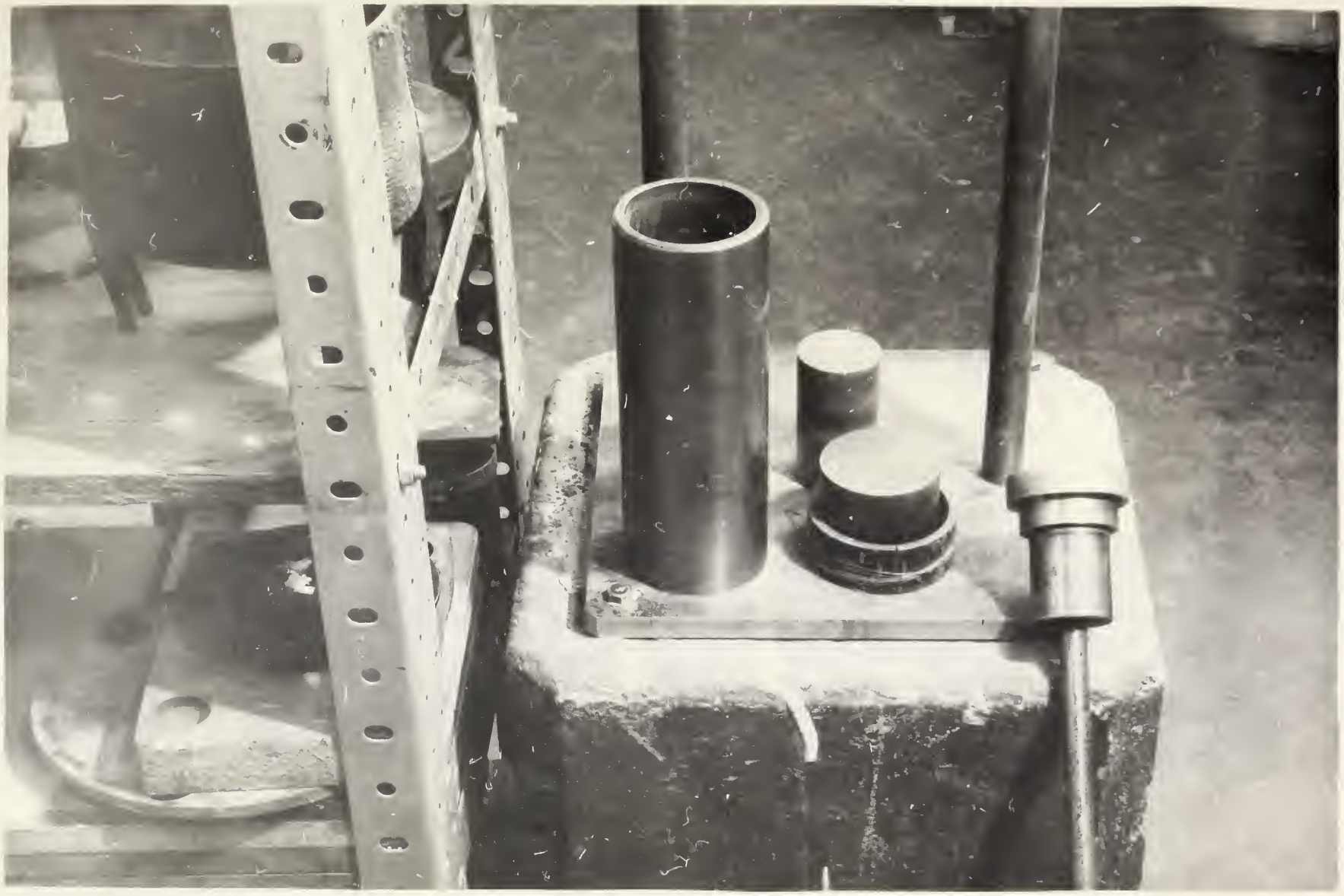


PHOTO I. COMPACTING BLOCK, MOLD, BASE AND HAMMER

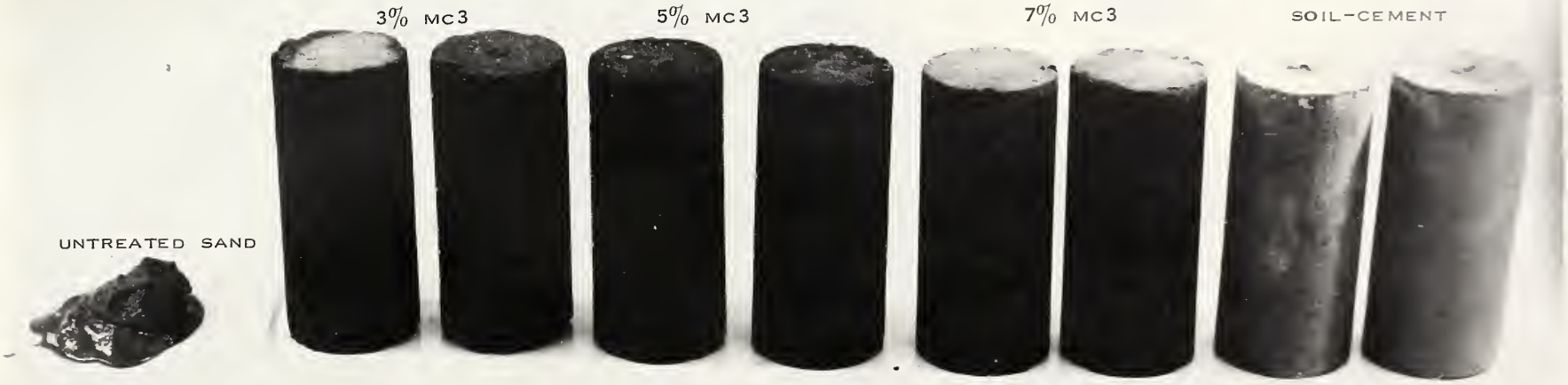


PHOTO 2. SAND-ASPHALT AND SOIL-CEMENT SPECIMENS AFTER IMMERSION

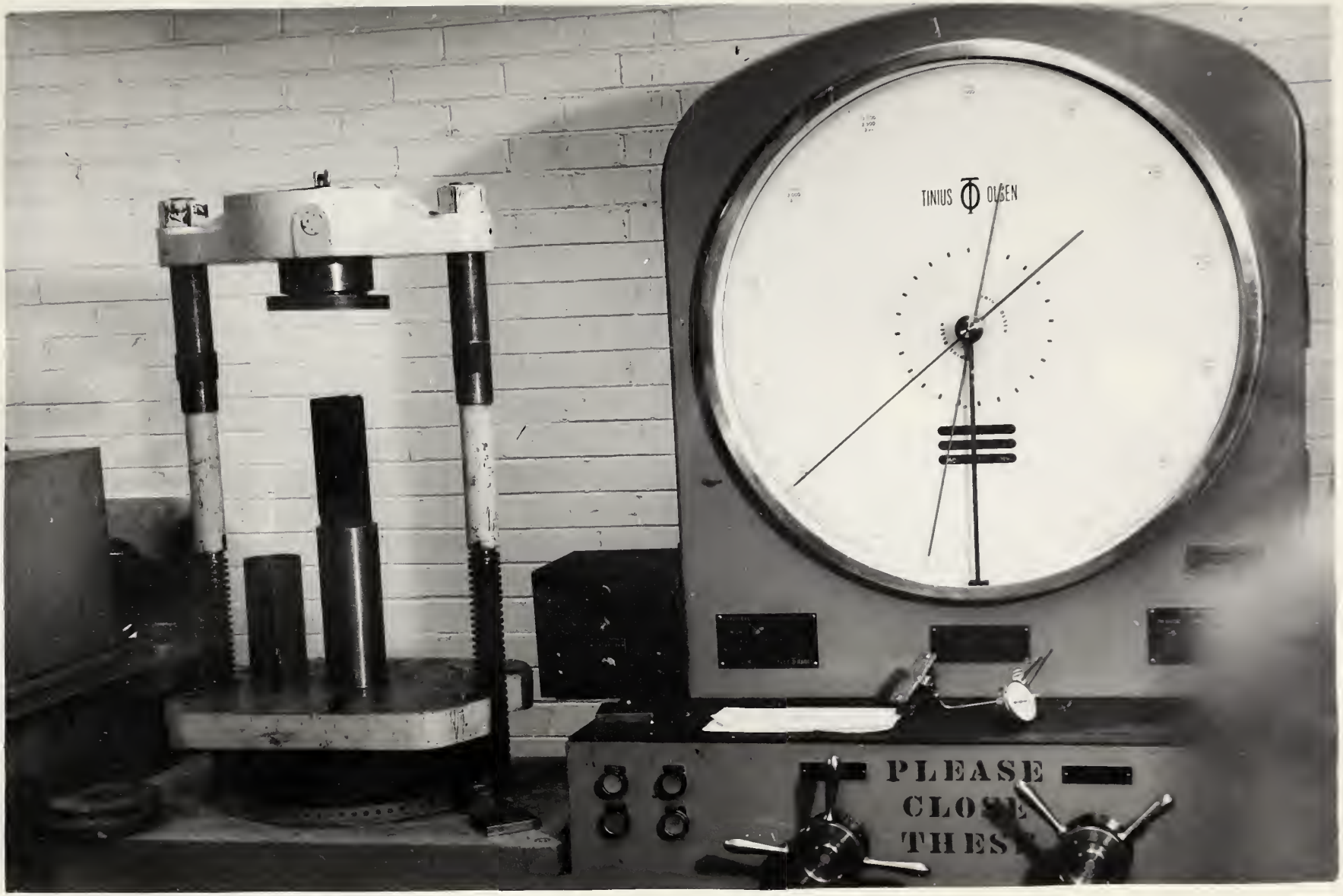


PHOTO 3. EXTRUDING USING TINIUS OLSEN MACHINE

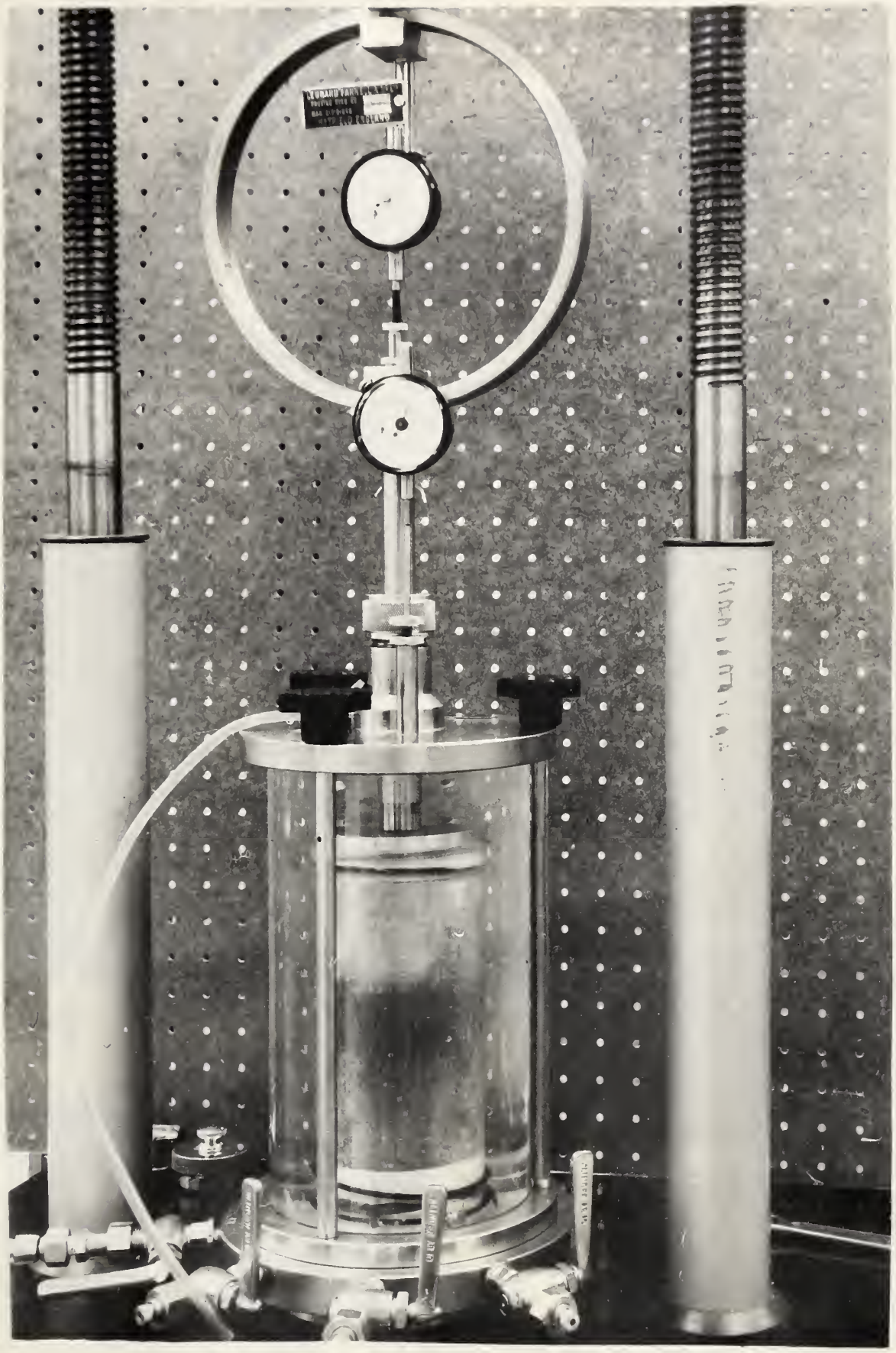


PHOTO 4. SPECIMEN READY FOR TRIAXIAL COMPRESSION

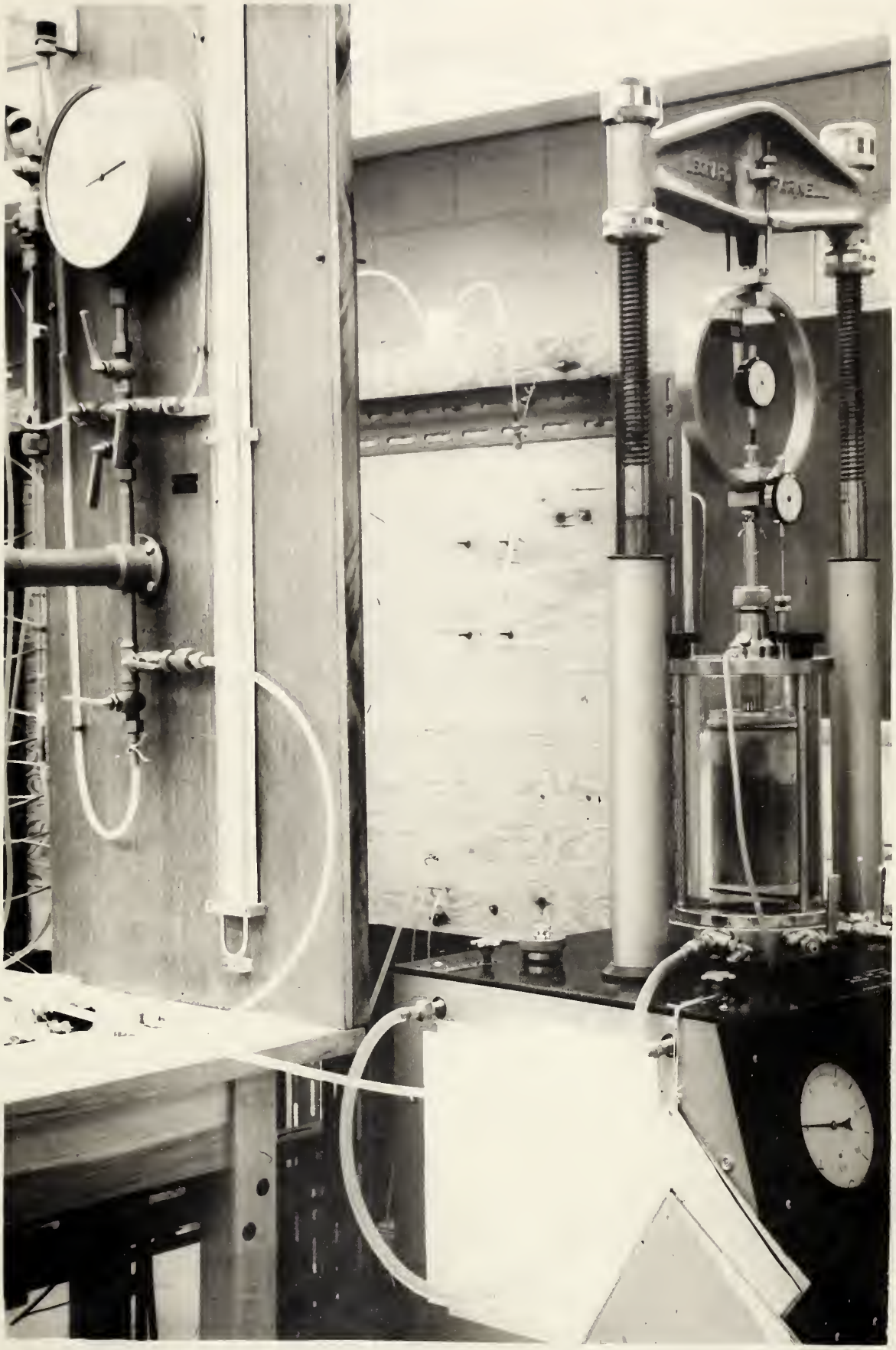


PHOTO 5. LATERAL PRESSURE APPARATUS AND FARNELL TESTER

CHAPTER V

SUMMARY OF RESULTS

Figures 1 and 2 show the results obtained for the determination of optimum volatile content for the sand-asphalt triaxial specimens. Figure 1 is a plot of dry density of the total mix (based on asphalt and soil solids) versus molding volatile content (Based on dry weight of soil). Figure 2 is a plot of unconfined compressive strength versus molding volatile content.

Figure 3 is a plot of friction angle and cohesion versus MC3 asphalt content for both immersed and cured specimens. Figure 4 is a plot of the ten Mohr rupture envelopes obtained from the testing program. Although the Mohr envelopes were assumed linear in the range of the confining pressures employed, it may not necessarily be correct to extrapolate the envelopes to higher failure stresses. At the higher failure stresses the envelope may become curved, although this could not be determined from this investigation. The envelopes of Figure 4 have been extrapolated beyond the failure stress conditions.

Figures 5, 6, and 7 are stress-strain curves obtained from the triaxial tests with a lateral pressure of 20 pounds per square inch. These typical stress-strain curves are the average of three specimens obtained by drawing a average curve for the three individual stress-strain curves.

Table 4 is a summary of results for the determination of the optimum volatile content for the sand-asphalt specimens. Tables 5 to 9 inclusive, are the triaxial results of the cured sand-asphalt and

immersed sand-asphalt and the soil-cement specimens. In these results the voids and dry density are expressed as a percentage of the soil solid volume and also of the soil solid volume and asphalt residue volume. The specific gravity of the residue asphalt was assumed equal to one. All results are the average of three specimens which were tested at each particular lateral pressure. For all specimens the deviator stress (S_1 - S_3) dropped at failure and the strain at the point of maximum deviator stress is recorded as the failure strain.

The moduli of deformation were obtained on the basis of the initial tangent modulus. Most specimens had a characteristic straight-line portion of the stress-strain curve at low strains.

The friction angle, cohesion and confidence limits were all obtained by the analytic method of Balmer (7). The confidence limits are tabulated for an arbitrary range of five normal stresses with corresponding shear stresses. Sample calculations for friction angle, cohesion and confidence limits are given in Appendix A.

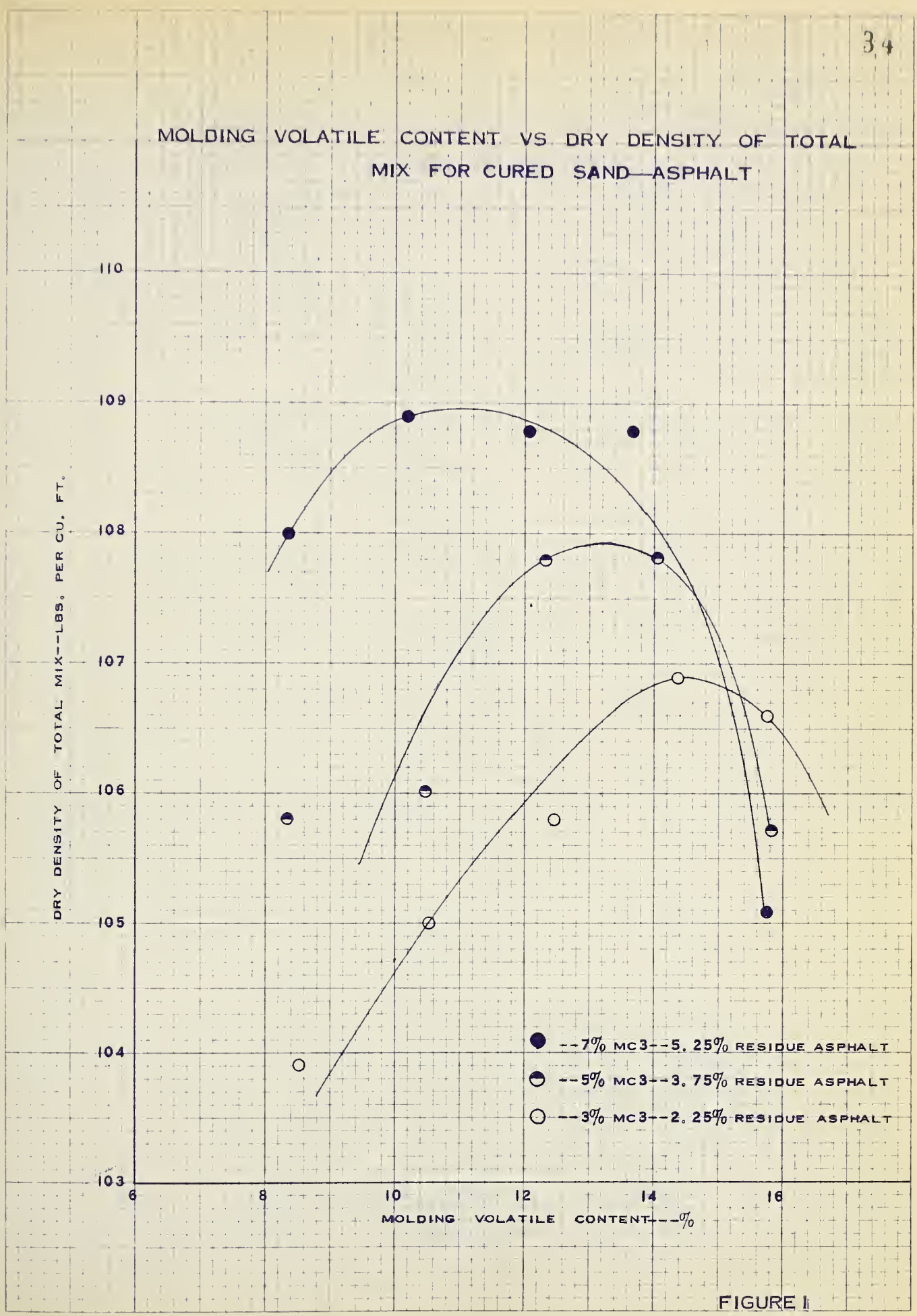

MOLDING VOLATILE CONTENT VS DRY DENSITY OF TOTAL MIX FOR CURED SAND—ASPHALT
DRY DENSITY OF TOTAL MIX--LBS. PER CU. FT.
110
109
108
107
106
105
104
103
6
8
10
12
14
16
MOLDING VOLATILE CONTENT---%
--7% MC3--5.25% RESIDUE ASPHALT
--5% MC3--3.75% RESIDUE ASPHALT
--3% MC3--2.25% RESIDUE ASPHALT

FIGURE I

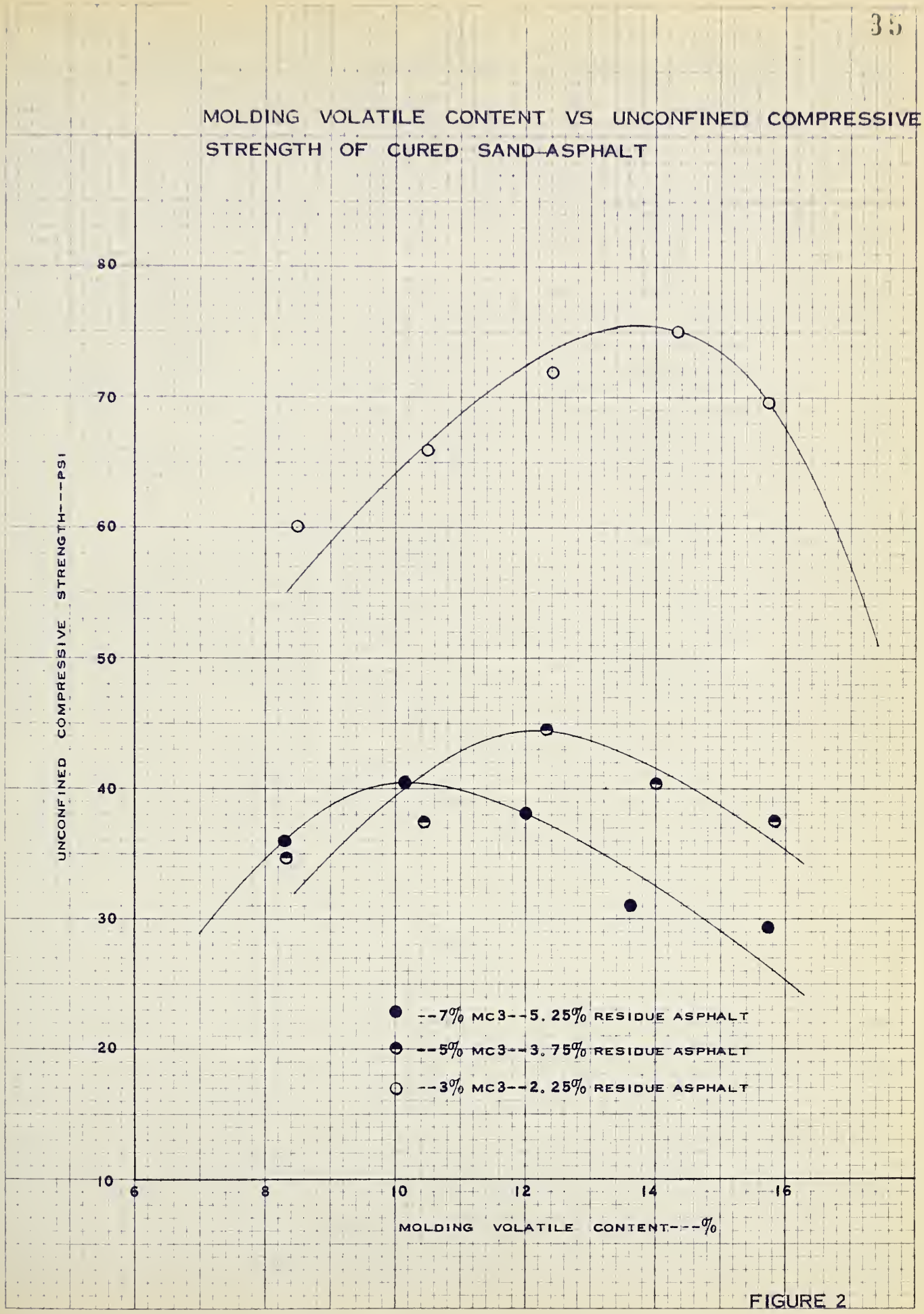

MOLDING VOLATILE CONTENT VS UNCONFINED COMPRESSIVE STRENGTH OF CURED SAND-ASPHALT
UNCONFINED COMPRESSIVE STRENGTH---PSI
80
70
60
50
40
30
20
10
6
8
10
12
14
16
MOLDING VOLATILE CONTENT---%
--7% MC3--5.25% RESIDUE ASPHALT
--5% MC3--3.75% RESIDUE ASPHALT
--3% MC3--2.25% RESIDUE ASPHALT

FIGURE 2

ASPHALT CONTENT VERSUS FRICTION ANGLE AND COHESION FOR CURED AND IMMERSED SAND-ASPHALT SPECIMENS

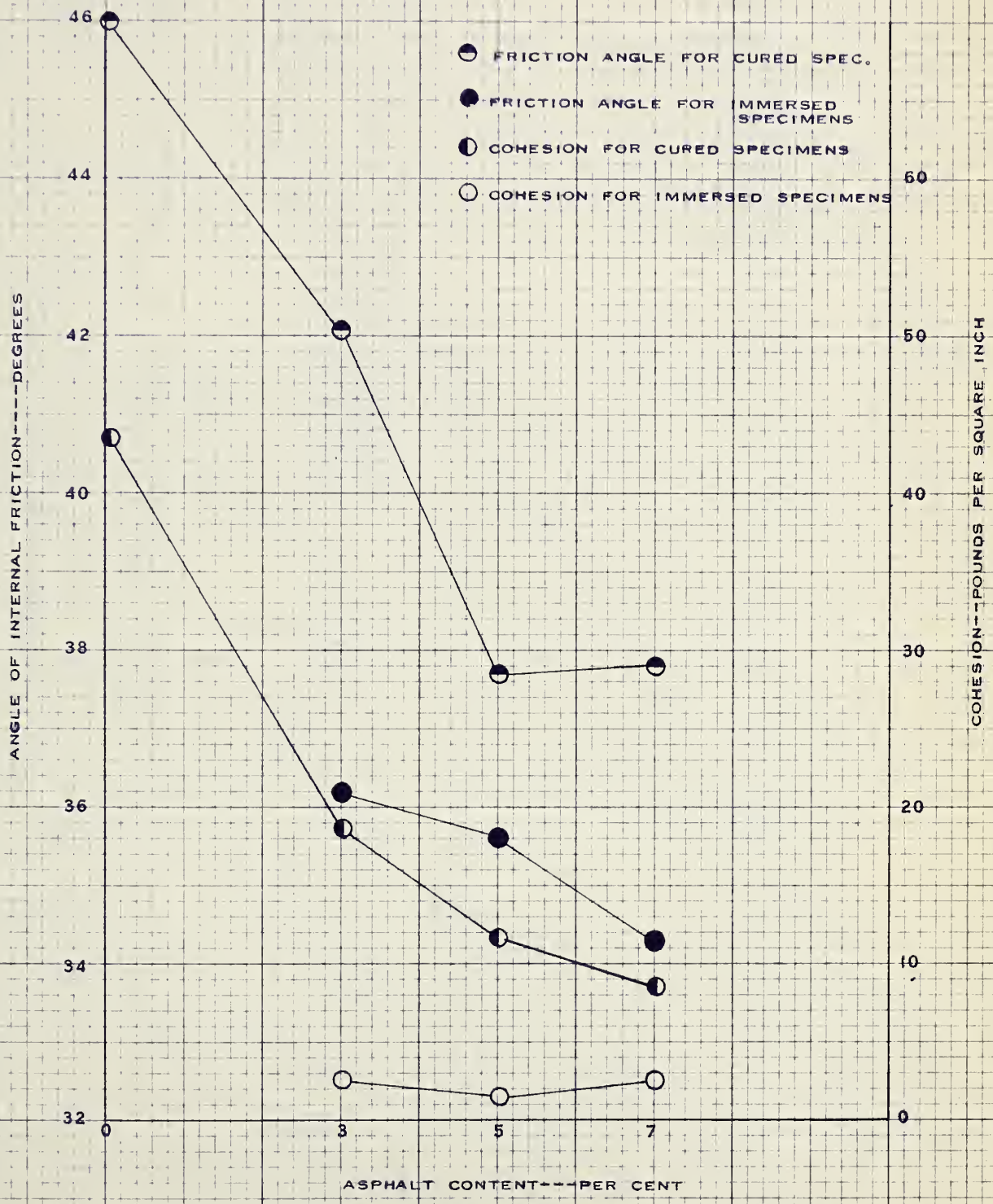
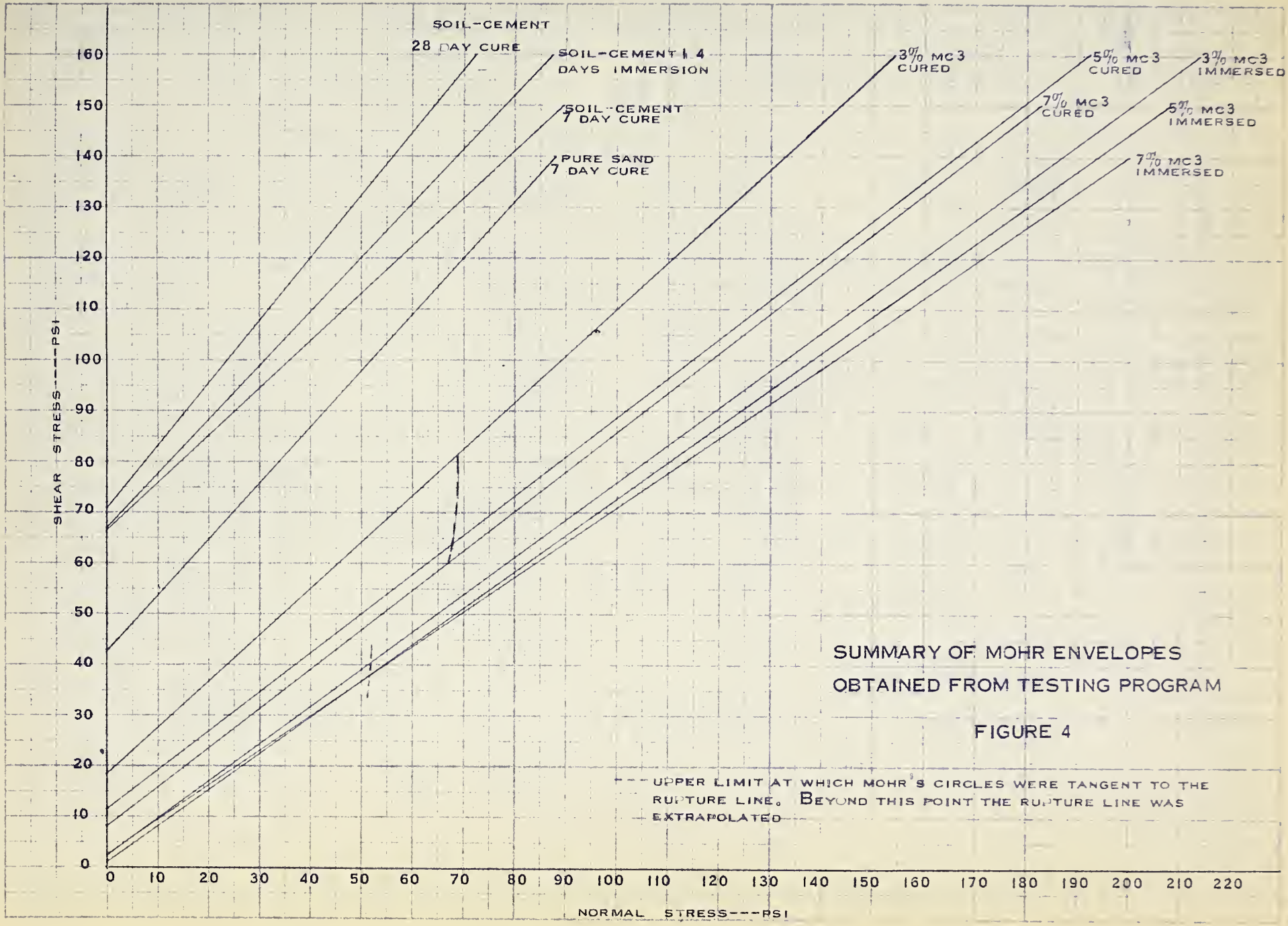


FIGURE 3



SUMMARY OF MOHR ENVELOPES OBTAINED FROM TESTING PROGRAM

FIGURE 4

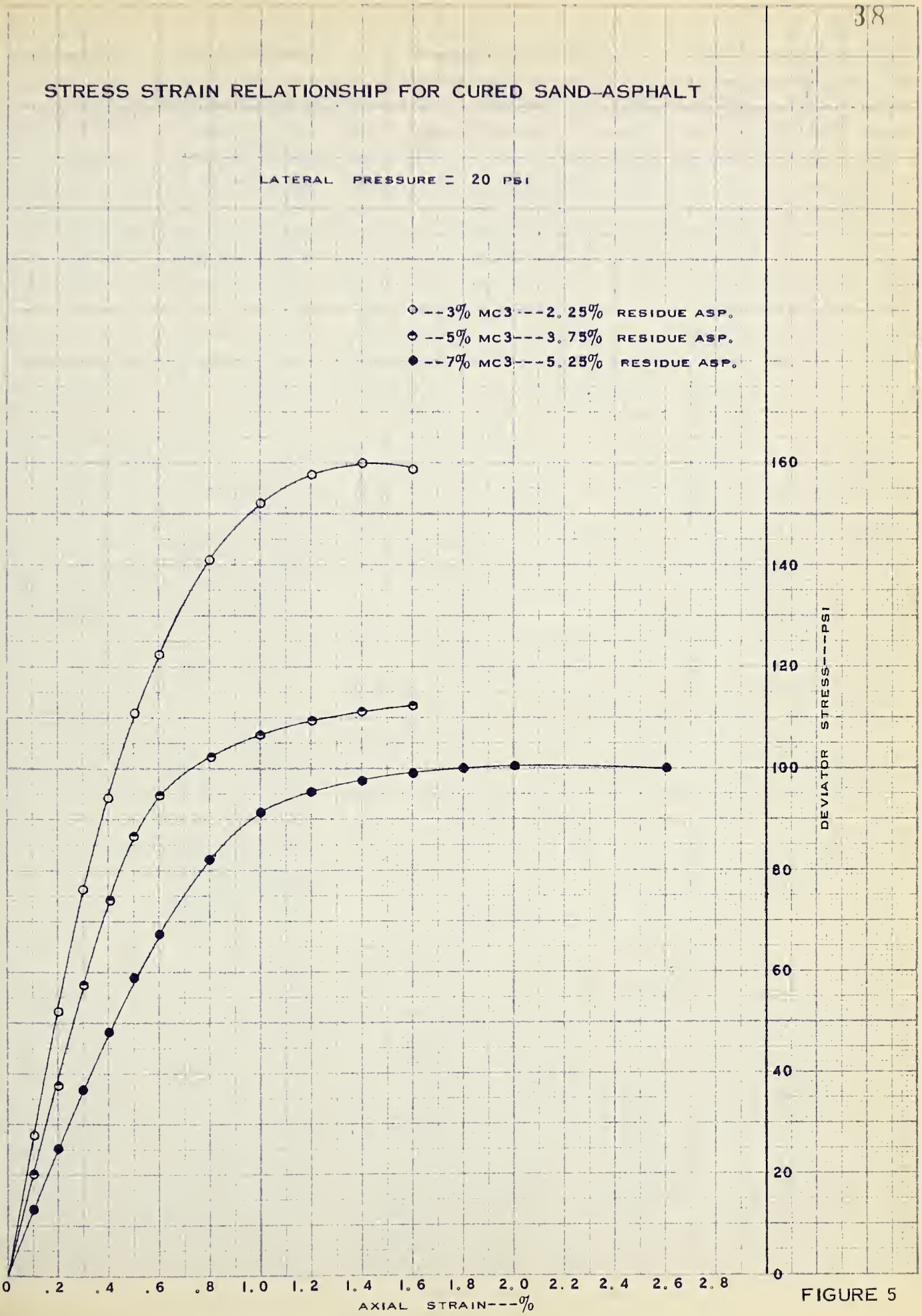

STRESS STRAIN RELATIONSHIP FOR CURED SAND-ASPHALT
LATERAL PRESSURE = 20 PSI
--3% MC3---2.25% RESIDUE ASP.
--5% MC3---3.75% RESIDUE ASP.
--7% MC3---5.25% RESIDUE ASP.
DEVIATOR STRESS---PSI
0
20
40
60
80
100
120
140
160
0
.2
.4
.6
.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
AXIAL STRAIN---%

FIGURE 5

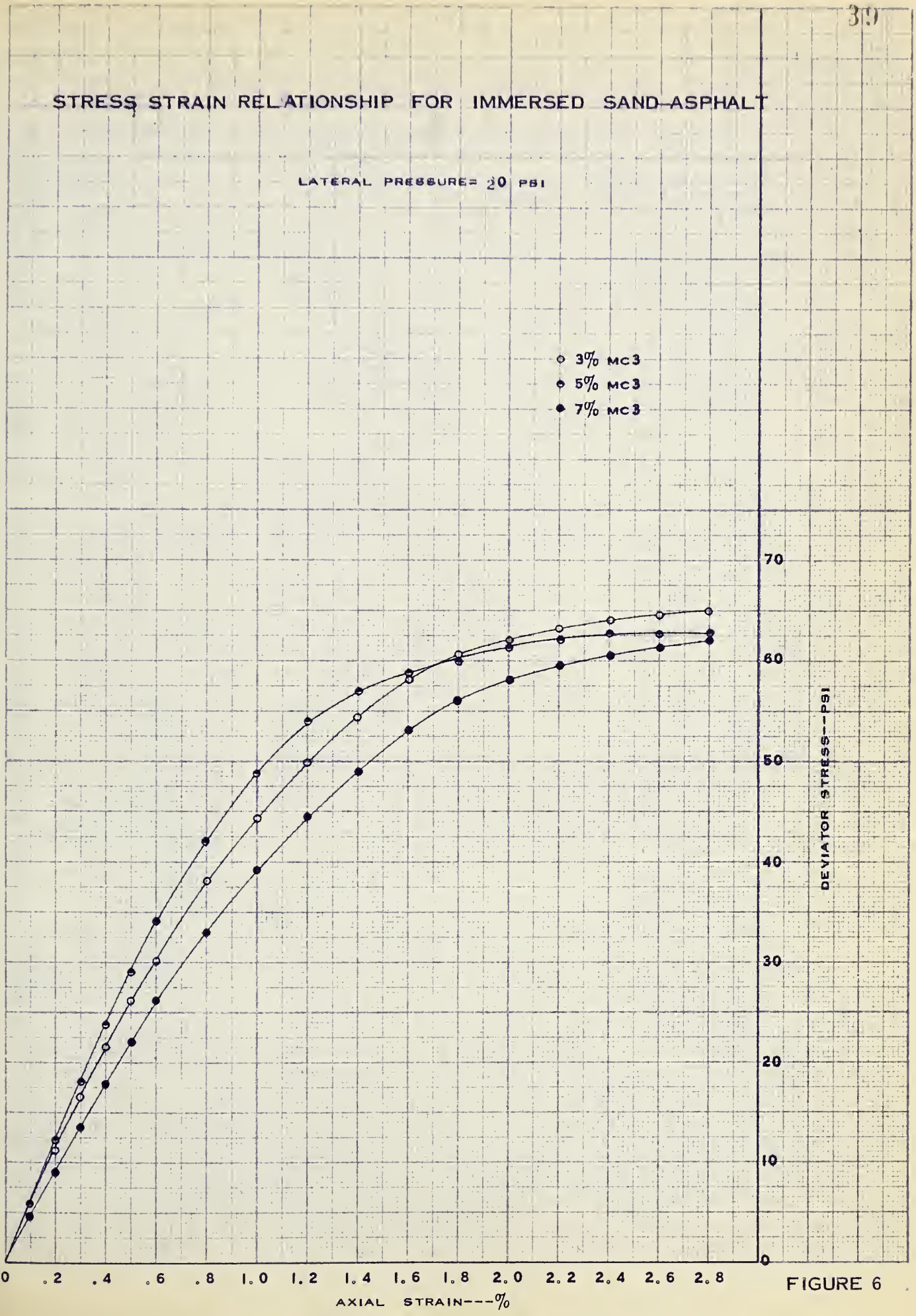

STRESS STRAIN RELATIONSHIP FOR IMMERSED SAND-ASPHALT
LATERAL PRESSURE= 20 PSI
3% MC3
5% MC3
7% MC3
DEVIATOR STRESS--PSI
0
10
20
30
40
50
60
70
0
.2
.4
.6
.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
AXIAL STRAIN---%

FIGURE 6

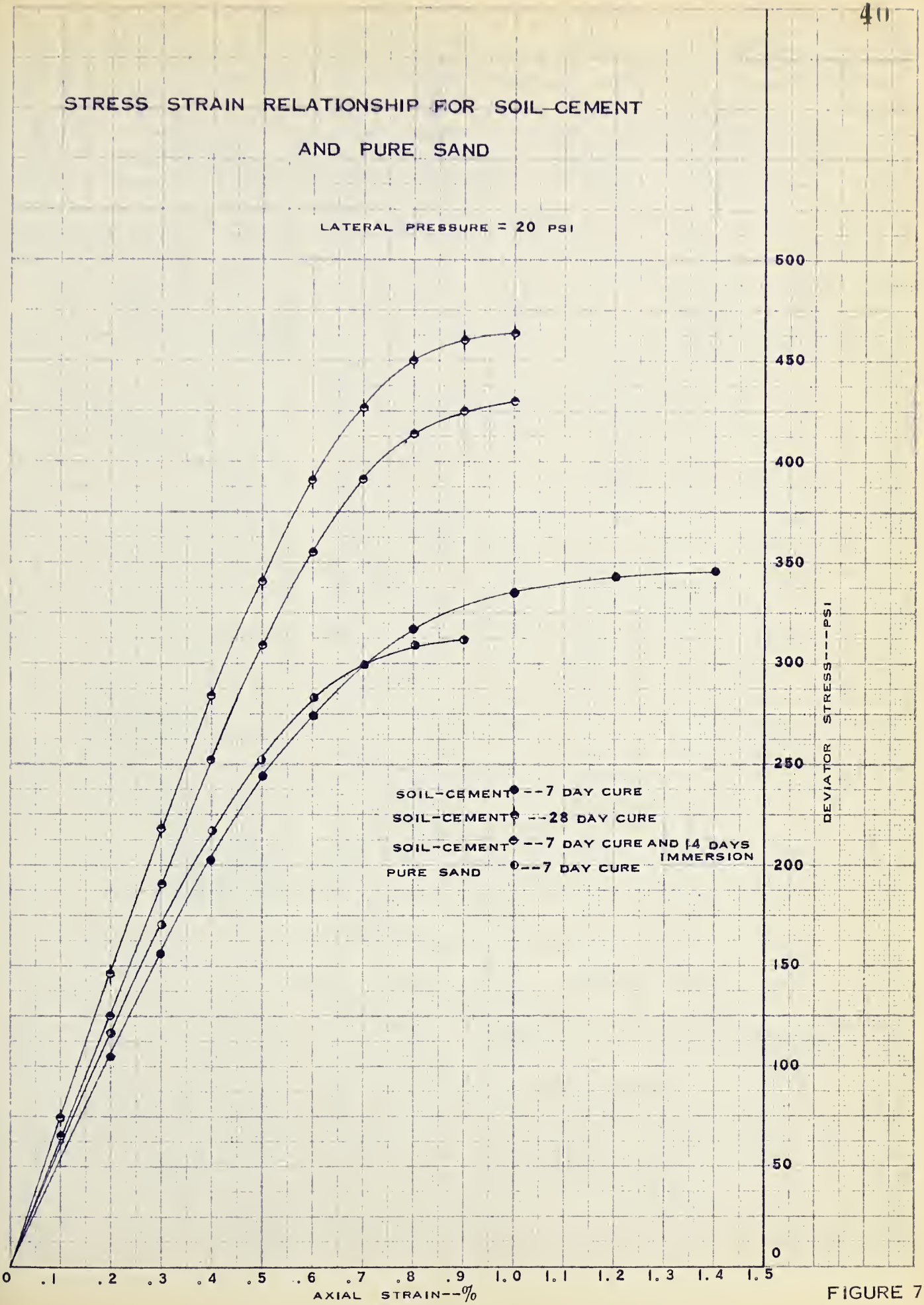

STRESS STRAIN RELATIONSHIP FOR SOIL-CEMENT
AND PURE SAND
LATERAL PRESSURE = 20 PSI
SOIL-CEMENT --7 DAY CURE
SOIL-CEMENT --28 DAY CURE
SOIL-CEMENT --7 DAY CURE AND 14 DAYS IMMERSION
PURE SAND --7 DAY CURE
DEVIATOR STRESS--PSI
0
50
100
150
200
250
300
350
400
450
500
0
.1
.2
.3
.4
.5
.6
.7
.8
.9
1.0
1.1
1.2
1.3
1.4
1.5
AXIAL STRAIN--%

FIGURE 7

TABLE 4

RESULTS OF MOLDING VOLATILE CONTENT VERSUS DENSITY AND UNCONFINED COMPRESSIVE STRENGTH

(Specimens Cured 7 Days at 100 Degrees F.)

% Asp	Mold. Volat Cont-ent	Weight			Vol.	Voids			Dry Density		Un-con Comp.
		Dry Samp-le	Before Test	Asp.	Voids	% Fill. Asp.	% of Soil Solid	% of Total Mix	Soil Solid pcf	Total Mix pcf.	psi
5*	15.8	1083	1093	35	247**	14.2	62.8	38.6	102.2	105.7	37.4
5	14.0	1108	1119	36	238	15.1	59.3	37.2	104.3	107.8	41.3
5	12.3	1107	1117	36	238	15.1	59.3	37.2	104.3	107.8	44.7
5	10.4	1089	1099	37	245	15.1	62.1	38.3	102.8	106.0	37.6
5	8.3	1087	1097	36	246	14.6	62.4	38.4	102.5	105.8	35.0
3	15.7	1094	1102	24	239	10.0	59.6	37.4	104.2	106.8	69.9
3	14.3	1096	1104	25	239	10.5	59.6	37.4	104.4	106.9	75.2
3	12.4	1082	1090	22	243	9.1	61.2	38.0	103.2	105.8	72.1
3	10.5	1076	1085	25	246	10.2	62.4	38.4	102.6	105.0	66.0
3	8.5	1066	1073	24	250	9.6	64.2	39.1	101.6	103.9	60.0
7	15.7	1079	1090	54	256	21.0	66.6	40.0	100.0	105.1	29.5
7	13.6	1115	1124	56	244	23.0	61.6	38.2	103.1	108.8	31.2
7	12.0	1117	1127	57	243	23.4	61.2	38.0	103.3	108.8	38.1
7	10.1	1116	1125	56	243	23.0	61.2	38.0	103.3	108.9	40.5
7	8.3	1110	1121	57	245	23.2	62.0	38.3	103.0	108.0	35.7

*Each result is the average of three specimens

**Total Volume of Sample assumed constant at 640 c.c.

TABLE 5

DATA AND RESULTS OF CURED SAND-ASPHALT TRIAXIAL SPECIMENS

(Specimens Cured 7 Days at 100 Degrees F.)

% Add.	Lat. Pres	Weight			Vol. Voids	Voids			Dry Density	
		Sample Dry	Before Test	Asp.		% Fill Asp.	% Soil Solids	% Total Mix	Soil Solids pcf	Total Mix pcf
5*	30	1106	1114	39	**241	16.2	60.4	37.7	103.9	108.0
5	20	1112	1122	39	238	16.4	59.2	37.2	104.7	108.5
5	10	1114	1122	39	237	16.0	58.8	37.1	104.9	108.8
5	0	1112	1121	38	238	16.8	59.4	37.2	104.6	108.2
5	Aver.	1111	1120	39	238	16.4	59.2	37.2	104.3	108.3
3	30	1095	1103	25	239	10.4	59.6	37.4	104.1	106.8
3	20	1096	1104	25	239	10.4	59.6	37.4	104.1	106.8
3	10	1098	1104	27	239	11.3	59.6	37.4	104.5	106.9
3	0	1098	1103	26	239	10.8	59.3	37.2	104.5	106.9
3	Aver.	1097	1104	26	239	10.8	59.6	37.4	104.2	106.8
7	30	1123	1133	55	240	22.9	60.0	37.5	104.1	109.5
7	20	1122	1132	56	241	23.2	60.4	37.7	103.9	109.5
7	10	1123	1133	56	240	23.3	60.0	37.5	104.2	109.3
7	0	1119	1128	57	242	23.6	60.7	37.8	103.6	108.9
7	Aver.	1122	1132	56	241	23.2	60.4	37.7	103.9	109.2
0	30	1063	1067	0	242	0	60.7	-	104.0	-
0	20	1069	1073	0	240	0	60.0	-	104.1	-
0	10	1069	1073	0	240	0	60.0	-	104.1	-
0	0	1064	1068	0	242	0	60.7	-	104.0	-
0	Ave	1066	1070	0	241	0	60.5	-	104.0	-

*Each result is the average of three specimens
**Total Volume of sample assumed constant at 640 c.c.

TABLE 6

RESULTS OF CURED SAND-ASPHALT TRIAXIAL SPECIMENS

(Specimens cured 7 days at 100 degrees F.)

% Add	Lat. Pres psi	Failure Strain %	Deviator Stress psi	Modulus of Deformation psi	ϕ Deg.	C psi	Confidence Limits: X Normal Stress	Y Shear Stress	2 Stan. Dev.	Y + 2σ	Y - 2σ
5	30	2.1	141	13,500	37.7	11.9	0	11.9	1.93	13.8	10.0
5	20	1.6	111	17,650			34.3	38.3	1.66	40.0	36.6
5	10	1.1	83	12,466			64.3	61.5	1.83	63.3	59.7
5	0	0.8	46	7,200			94.3	84.5	2.47	87.0	82.0
							154.3	130.9	3.58	134.5	127.3
3	30	1.9	191	24,890	42.1	18.7	0	18.7	2.83	21.5	15.9
3	20	1.3	158	23,200			36.9	52.1	1.28	53.4	50.8
3	10	1.0	138	22,270			66.9	79.2	2.42	81.6	76.8
3	0	0.8	88	16,900			96.9	107.3	4.30	111.6	103.0
							156.9	160.7	8.32	169.0	152.4
7	30	3.2	127	13,950	37.8	8.6	0	8.6	1.41	10.0	7.2
7	20	2.3	105	11,700			31.2	32.8	0.70	33.5	32.1
7	10	1.3	65	11,230			61.2	56.0	1.36	57.4	54.6
7	0	0.8	35	5,880			91.2	79.2	2.46	81.7	76.7
							151.2	125.8	4.76	130.6	121.0
0	30	1.3	323	48,660	46.0	43.2	0	43.2	6.55	49.8	36.6
0	20	0.9	311	55,700			52.3	97.7	2.24	99.9	95.5
0	10	0.8	242	45,800			72.3	118.4	3.24	121.6	115.2
0	0	0.7	182	30,000			92.3	139.2	5.20	144.4	134.0
							102.3	150.0	6.30	156.3	143.7

TABLE 7

DATA AND RESULTS OF IMMERSED SAND-ASPHALT TRIAXIAL SPECIMENS

(Specimens Cured 7 Days at 100 Degrees F. and Immersed for 14 Days)

% Add	Lat. Pressure	Weight			Vol. Voids	Voids				Dry Density	
		Dry Sample	Before Test	Asp.		% Filled Asp.	% Filled Water	% of Soil Solid	% of Total Mix	Soil Solids pcf	Total Mix pcf
5*	30	1104	1237	38	240**	15.8	55.4	60.0	37.4	104.1	107.8
5	20	1107	1227	37	239	15.5	50.2	59.6	37.4	104.1	107.9
5	10	1110	1227	38	238	16.0	49.2	59.2	37.2	104.6	108.2
5	0	1112	1226	36	237	15.2	48.0	59.0	37.0	104.8	108.3
5	Aver.	1108	1229	37	238	15.5	50.8	62.4	37.2	104.6	107.9
3	30	1102	1227	24	236	10.2	53.0	58.4	36.8	105.0	107.3
3	20	1104	1223	25	236	10.6	50.4	58.4	36.8	105.2	107.8
3	10	1101	1218	24	236	10.2	49.6	58.4	36.8	104.8	107.2
3	0	1098	1216	26	238	10.9	49.6	59.2	37.2	104.5	107.0
3	Aver.	1101	1221	24	236	10.9	50.8	58.4	36.8	104.8	107.2
7	30	1122	1226	56	240	23.4	43.3	60.0	37.5	103.9	109.3
7	20	1124	1223	56	240	23.4	41.3	60.0	37.5	104.2	109.4
7	10	1123	1221	56	240	23.4	40.8	60.0	37.5	104.1	109.6
7	0	1121	1224	55	240	22.9	42.9	60.0	37.5	104.0	109.3
7	Aver.	1123	1224	56	240	23.4	42.0	60.0	37.5	104.1	109.6

*Each result is the average of three specimens
**Total Volume of sample assumed constant at 640 c.c.

TABLE 8

RESULTS OF IMMERSED SAND-ASPHALT TRIAXIAL SPECIMENS

(Specimens Cured 7 Days at 100 Degrees F. and Immersed for 14 days)

% Add	Lat. Pres (psi)	Failure Strain (%)	Deviator Stress (psi)	Modulus of Deformation (psi)	ϕ Deg.	C psi	Confidence Limits X Normal Stress	Y Shear Stress	2 Std. Dev.	Y+ 2σ	Y- 2σ
5*	30	3.3	88.4	6200	35.6	1.5	0	1.5	2.3	3.8	-0.8
5	20	2.5	63.4	5780			25	19.4	0.5	19.9	18.9
5	10	2.3	31.9	2470			45	33.7	1.8	35.5	31.9
5	0	1.3	6.4	780			65	48.0	3.6	51.6	44.4
							115	83.6	8.0	91.6	75.6
3	30	2.9	98.8	8000	36.2	2.7	0	2.7	0.9	3.6	1.8
3	20	2.8	65.7	5400			26.2	21.9	0.5	22.4	21.4
3	10	2.2	42.3	4900			46.2	36.5	0.8	37.3	35.7
3	0	1.7	10.0	850			66.2	51.1	1.2	52.3	49.9
							126.2	95.1	2.9	98.0	92.2
7	30	3.5	87.2	5800	34.3	2.8	0	2.8	0.4	3.2	2.4
7	20	3.1	63.7	4400			25.8	20.4	0.2	20.6	20.2
7	10	2.6	38.1	2800			45.8	34.2	0.3	34.5	33.9
7	0	2.1	9.2	560			65.8	47.8	0.6	48.4	47.2
							125.8	88.8	1.8	90.6	87.0

*Each result is the average of three specimens

TABLE 2

RESULTS OF SOIL-CEMENT TRIAXIAL SPECIMENS

S.C. Series	Lat. Pres. psi	Failure Strain %	Deviator Stress psi	Modulus of Deformation psi	ϕ Deg.	C psi	Confidence Limits X Normal Stress	Y Shear Stress	2 Stan Dev.	Y + 2σ	Y- 2σ
7 Day Cure	80	2.6	554	64,580	39.7	66.5	0	66.5	4.8	71.3	61.7
	50	2.6	464	40,370			63.0	118.5	2.7	121.2	115.8
	20	1.5	345	50,250			113.0	159.7	1.9	161.6	157.8
	0	0.8	285	44,765			133.0	176.5	2.1	178.6	174.4
							153.0	192.5	2.5	195.0	190.0
7 Day Cure & 14 Days Soak	80	2.6	677	70,150	44.0	67.0	0	67.0	5.7	72.7	61.3
	50	1.7	539	65,600			62.0	126.7	3.3	130.0	123.4
	20	1.0	424	62,000			112.0	174.8	2.3	177.1	172.5
	0	0.8	307	45,000			142.0	204.0	2.7	206.7	201.3
							162.0	223.0	3.3	226.3	219.7
28 Day Cure	80	-	Not	Performed			0	71.0	9.9	80.9	61.1
	50	1.7	638	75,330	48.0	71.0	35.5	110.4	6.6	117.0	103.8
	20	1.0	453	71,700			85.5	166.0	3.8	169.8	162.2
	0	0.9	360	57,030			115.5	199.2	5.0	204.2	194.2
							135.5	221.5	6.6	228.1	214.9

Average dry density at compaction = 109.8 lbs./cu. ft.
Average dry density at testing = 111.5 lbs./cu. ft.
(for all specimens)
Average wt. of all specimens at compaction= 1267 gms.
Average wt. at testing of 7 & 28 cured series=1292 gms.
Average wt. at testing of 14 day immersed series= 1319 gms.
Average dry wt. of all specimens = 1144 gms.

CHAPTER VI

DISCUSSION OF RESULTS

The optimum molding volatile content for a sand-asphalt admixture tends to decline for increasing amounts of cutback asphalt. The optimum volatile contents for optimum density and unconfined strength are summarized below from Figures 1 and 2. The pure sand data was obtained from Table 5 and Appendix B.

% Residue	Optimum Vol. Content for Density	Max. Dry Density (lbs/ft^3)	Uncon. Comp. (psi)	Opt. Vol. Content for U. C. S.
0	15.5	104.0	180	15.5
2.25	14.4	106.9	76	13.8
3.75	13.2	107.9	44	12.4
5.25	11.0	109.0	41	10.4

Examination of the above data indicates that the residue asphalt is effective as a lubricating agent and tends to reduce the volatiles required for maximum density accomplishment. The three residue percentages of 2.25, 3.75, and 5.25 are as effective as water in the ratio of 0.49, 0.62, and 0.84, respectively. For example, with 2.25 per cent residue asphalt the optimum volatile content is decreased to 14.4 per cent from 15.5 per cent for the untreated sand. This is a reduction of 1.1 per cent. Thus the 2.25 per cent asphalt is as effective a lubricate as 1.1 per cent water or 0.49 times as effective as the water.

The volatile contents for maximum unconfined strength are not exactly equal to those for maximum density. Katti (10) found that the optimum molding volatile content for strength, and density were not identical but were very near. Consequently, for all practical purposes of design, the volatile content for maximum density can be concluded to be that required to also maintain maximum strength.

Upon visual inspection of Figure 2 it can be realized that with increasing percentages of asphalt the strength appears to become less dependent on molding volatile content; this trend is coincident with the density variation which is also less dependent on molding water content at higher asphalt percentages.

The triaxial testing program resulted in ten Mohr rupture envelopes which are shown in Figure 4. It must be emphasized that the results are based on testing of specimens in a very limited range of curing and immersion conditions. The sand-asphalt specimens could be tested under more severe conditions by allowing immersion before complete curing.

The effects of asphalt on the cured strength of the mix, are found to reduce both the angle of internal friction and the cohesion. The loss in strength of the cured specimens increases with increase in asphalt content, although the 5 and 7 per cent MC3 mixes gave Mohr envelopes near identical. The asphalt tends to act as a lubricant.

reducing the friction angle and the cohesion. With the addition of 3 per cent MC3 the cohesion is halved from 43 pounds per square inch to 18.7 pounds per square inch and the angle of internal friction is decreased from 46.0 degrees to 42.0 degrees (See Table 6).

The loss in strength of the cured specimens due to the presence of the asphalt may be explained by the forces which act between the soil particles. After water has evaporated from a soil, high surface tension forces are built up which produce high apparent cohesion. Consequently, as long as the material depends on the soil to soil bonds, the strength is high but with the addition of asphalt some of the soil to soil bonds are replaced by asphalt to soil bonds. If these asphalt-soil bonds are not as strong as the soil to soil particle bonds there is a reduction in cured strength (5).

Of course, the stabilizing of a soil with asphalt is to maintain the immersed strength of the material. This sand which was stabilized with MC3 does maintain strength upon immersion although there is a considerable drop in cohesion and angle of internal friction from the cured condition. The shear strength of the mixes at the percentages of asphalt employed show that the immersed strength also dropped with increasing asphalt content. Although testing was performed after fourteen days immersion, some specimens were soaked for periods of thirty days. Water continued to be absorbed at thirty days, but there was little visible deterioration of the specimens. By examining the results obtained from the immersed and cured sand-asphalt specimens it could be concluded that asphalt stabilization has no

beneficial effects. But upon immersion of three untreated sand specimens which had been cured for seven days, disintegration occurred within five minutes. Although the shear strength is reduced by the addition of asphalt, the sand is given cohesion under immersed conditions. The immersed strength was greatest for the 3 per cent MC3 with a residue of 2.25 per cent. The optimum is therefore somewhere between zero and 2.25 per cent asphalt. For field construction due to poor mixing as compared to laboratory mixing it would be suggested that a MC3 asphalt content of four per cent be used, thus leaving a residue of 3.20 per cent.

The major role of the asphalt in stabilizing of this sand is to give the sand cohesion. Although the asphalt prevents the entrance of water, the ultimate water absorbed would be sufficient to cause crumbling of the material if the cohesion were not present.

An explanation of the reduction in strength with increasing asphalt content upon immersion cannot be postulated as due to the amount of water contained in the voids. The 7 per cent mix contained the least water but also gave the lowest strength. The total volume of voids appeared to be independent of asphalt percentage. The only variation in void properties of the different immersed specimens was the air-void content. The 3, 5, and 7 per cent mixes had air-void contents of 39.5, 33.7, and 34.6 per cent, respectively (See Table 7). Hence, the 5 and 7 per cent mixes had voids filled with greater portions of asphalt and water. If the asphalt acts like a fluid, it can contribute to strength loss similar to that of water.

The immersion test is very severe for a base course material. The process by which a base course usually takes up moisture is by capillary rise of water from the water table. From many observations of highway and airport base courses it has been reported by McLeod (20) that non-waterproofed stabilized base courses reach a equilibrium moisture content of 4 to 7 per cent. For the same material stabilized with 1 to 2 per cent asphalt the equilibrium moisture content was found to be 1 to 3 per cent. These equilibrium moisture contents are for a conventional base course and may vary for a sandy material stabilized with asphalt. The moisture content of the specimens after immersion in this investigation was 11 per cent. Thus, it would appear to be most reasonable to base a design of the sand-asphalt on specimens near the cured strength or in a slightly wetter condition. But a small amount of water may still destroy the high apparent cohesion which was developed upon desiccation.

The moduli of deformation which were obtained for the soil-cement and sand-asphalt have significant implications. To compare the moduli, the minor principal stress of 20 pounds per square inch is selected. The soil-cement cured for seven days produced a modulus of 50, 250 pounds per square inch; whereas, the cured and immersed 3 per cent sand-asphalt had deformation moduli of 22, 270 and 4900 pounds per square inch, respectively. The soil-cement modulus increased to 71, 700

pounds per square inch for the twenty-eight day cured series. Probably, in the field the sand-asphalt would have a modulus somewhere between the ideal cured condition and the severe immersion condition. However, the soil-cement modulus would be at least five times that of the sand-asphalt. The higher deformation modulus indicates the degree to which the cement has improved the native material in terms of a higher failure stress at reduced strain. The relatively low modulus for the sand-asphalt compared to the soil-cement would result in much greater thicknesses of sand-asphalt for the same load bearing capacity. This was substantiated in Chapter VII by the design methods of Hank and Scrivner, the Kansas method and the Texas method.

Although soil-cement has a higher strength than the sand-asphalt, the cracking which results from shrinkage of the cement may limit the ultimate life of the soil-cement or increase the cost of maintenance.

The failures of the soil-cement specimens for all series was a cone type failure near the loading edge which then progressively resulted in a failure angle of sixty degrees to the horizontal. After the ultimate deviator stress was reached the load carrying capacity of the specimen dropped off rapidly. The average failure strain for the seven day cured soil-cement specimens at 20 pounds pounds per square inch lateral pressure was 1.5 per cent. The failures for the sand-asphalt cured series were similar to those of the soil-cement.

The failure strains at 20 pounds per square inch lateral pressure for the 3, 5, and7 per cent cured mixes were 1.3, 1.6 and 2.6 per cent, respectively. The failures of the immersed specimens were of the bulging type with no shear failure visible. The failure strains were increased for the three immersed asphalt contents above those of the cured specimens to 2.8, 2.5, and 3.1 per cent, respectively. For the immersed specimens it was possible to determine a maximum deviator stress but the load dropped off very slowly. Actually, the failure strains obtained in this investigation for the sand-asphalt were very low.

The cured sand-asphalt and the soil-cement failed as brittle materials; that is, after the maximum stress was reached the load dropped rapidly to zero. The immersed specimens were more flexible. The bulging type failures are an advantage because the load after being removed can be reapplied and maintained near its maximum value. Of course, immediately preceding failure of the soil-cement much cracking occurs and it is not possible to maintain a load after reapplication. Therefore, from this investigation the only structural advantage of the sand-asphalt is its greater flexibility as compared to that of the soil-cement.

Using twelve specimens for determination of each envelope resulted in confidence limits which showed very little variation at

any particular shear stress. The 95 per cent confidence limits are tabulated in Tables 6, 8 and 9 for various values of normal stress and shear stress.

It maybe of interest to compare the results of this investigation with that of work performed on other materials for base course application. Improvement of sub-marginal gravels with cement, lime and flyash was reported by Clark (21). Shear strengths of 55 pounds per square inch were available for a normal stress of 50 pounds per square inch. Thus it can be seen that even the immersed specimens with shear strengths of 40 pounds per square inch are competitive with the improved gravels. Results reported by Millions (22) on washing and crushing of sub-marginal gravels gave results similar to those of Clark.

The results of unconfined compression tests can be misleading when comparing materials of different load characteristics. From Figure 4 the shear strength of the soil-cement and sand-asphalt series can be determined for any normal stress on the plane of failure. Arbitrary normal stresses of 50 and 20 pounds per square inch are chosen and the corresponding shear stresses are compared to the unconfined compressive strengths of the different test series as follows:

	Shear Strength for Normal Stresses of (psi)		Unconfined Compressive Strength
	50	20	
Soil-cement 28 day cure	132	95	360
Soil-cement 14 day immersion	120	88	307
Soil-cement 7 day cure	113	85	285
Untreated sand	98	65	182
3% MC3 7 day cure	64	37	88
5% MC3 7 day cure	50	27	46
7% MC3 7 day cure	47	24	35
3% MC3 14 day immersion	39	18	10
5% MC3 14 day immersion	37	15	6.4
7% MC3 14 day immersion	37	16	9.2

Comparing the twenty-eight day cured soil-cement with the 7 per cent immersed sand-asphalt by means of the unconfined compressive strengths shows that the soil-cement has forty times the strength of the sand-asphalt but the actual shear strength available according to the triaxial test at a normal stress of 50 pounds per square inch is only four times the sand-asphalt. The superiority of the soil-cement is increased as the normal stress is reduced. The shortcomings and misconceptions of the unconfined test are definitely realized when compared with the triaxial test on materials of different load-deformation characteristics. The unconfined test

merely indicates that the soil-cement has superior structural properties but the degree of superiority is grossly exaggerated. The unconfined test would be valid if a base course was not subjected to any lateral support but this is not the case.

The Portland Cement Association (23) performed triaxial tests on soil-cement specimens and found that the angle of internal friction was relatively constant for variations in cement content and curing age. For sandy and silty soils treated with cement the average angle of internal friction found was 44 and 36 degrees, respectively. The corresponding values for the untreated soils were 36 and 27 degrees, respectively. It was also found that the cohesion tended to increase with curing time and increasing cement contents. Soaking specimens before testing also tended to lower the cohesion.

In this investigation there was a significant change in the angle of internal friction. The 7 day cure, 14 day immersion, and 28 day cure produced friction angles of 39.7, 44.0 and 48.0 degrees, respectively. The cohesion tended to vary less and the values were 66.5, 67.0, and 71.0 pounds per square inch, respectively. Actually under the triaxial conditions of this investigation there was little gain in shear strength from 7 day cure to 28 day cure. The 14 day immersion period did not lower the strength because the low permeability of the soil-cement prevented water absorption to a limited degree.

From the search of literature, there were no results available to compare the sand-asphalt parameters.

CHAPTER VII

ANALYSIS OF TEST RESULTS BY COMPARISON WITH VARIOUS METHODS OF BASE COURSE DESIGN

In Canada and United States only two methods of design and correlation of base materials are used today which utilize results of the triaxial tests. These two methods are employed by the states of Texas and Kansas.

A method of thickness design of highway materials based on Burmister's theory of stresses in layered systems has been formulated by Hank and Scrivner (14). The Mohr rupture line obtained in a laboratory triaxial test is applied using the two-layer system.

An attempt will be made to compare the soil-cement and sand-asphalt stabilization admixtures by the three methods outlined above.

Kansas Method

The method of design of the thickness of flexible pavements used in Kansas is based on the results of triaxial compression tests (6) (15) (16) (17).

This method is based on the theoretical stresses in ideal masses using the Boussinesq equations. The application to design was first proposed by Palmer and Barber (12). It was assumed that the pavement was incompressible and that the strains were elastic and took place only in the subgrade. The Boussinesq settlement equation can be expressed

as follows:

$$T = \sqrt{(3P/2\pi ES)^2 - a^2}$$

where;

T = pavement thickness

P = total load on single wheel

E = modulus of elasticity of subgrade

a = radius of contact

S = allowable deflection

To utilize the formula for design purposes it is necessary to assume a limiting deflection. The design deflection of the surface is assumed to be 0.1 inches for the flexible pavements. This value was determined from measurements of numerous flexible pavements in various conditions.

The design procedure calls for all samples to be tested in a saturated condition. Saturation is deemed desirable in order to obtain a direct comparison for different types of materials. To account for the possibility of the soil not being saturated after construction, a coefficient, n, is introduced which gives a decreased thickness of surface or base course required in localities of less rainfall. A coefficient, m, is also introduced to account for traffic intensity variation.

The equation formulated by Palmer and Barber with the Kansas modifications is as follows:

$$T = \left[\sqrt{(3Pmn/2\pi CS)^2 - a^2}\right] \left[\sqrt[3]{C/C_p}\right]$$

Where;

T = thickness of pavement

C_p = modulus of elasticity of pavement or surface course (15,000 psi)

C = modulus of elasticity of subgrade or subbase

P = wheel load

m = traffic coefficient based on volume of traffic

n = saturation coefficient based on rainfall

a = radius of area of tire contact corresponding to P

S = permitted deflection of surface

The stiffness factor $(C/C_p)^{1/3}$ was proposed on the basis of the stiffness factor for slabs and was checked against the elastic displacement due to a point load in a two-layer system. The stiffness factor takes into account the stress distributing qualities of the paving surface as related to those of the subgrade soil.

Charts are prepared from the formula for determination of the pavement mat thicknesses (C_p = 15,000 psi) required on any subgrade. Although the charts give the total thickness of pavement mat (asphaltic concrete), it is common to employ only a shallow depth of the mat or wearing surface. Thus, if it is desired to substitute some granular base-course material for a portion of the bituminous mat, the relationship can be determined on the basis of the stiffness factor, for the bituminous mat related to the base course. Thus, if a thickness of bituminous mat is assumed, the thickness of base-course material is that given by:

$$t_t = (T - t_p) \sqrt[3]{C_p/C_t}$$

Where;

t_t = thickness of base course (soil-cement or sand-asphalt)

T = thickness of bituminous mat (total obtained from design charts)

t_p = desired thickness of bituminous mat

C_p = modulus of elasticity of bituminous mat

The stress-strain curve obtained from the triaxial compression test is used for determining the modulus of elasticity of the material being tested. A lateral pressure of 20 pounds per square inch is commonly employed.

A summary of sample preparation and testing used in this method is as follows:

(1) Specimens of subgrade are either cut from undisturbed samples or disturbed samples are obtained from compaction at saturation under a static load. The specimens are usually 2.8 inch diameter by 8.0 inches long.

(2) Undisturbed subgrade specimens are tested in a triaxial apparatus using a confining pressure of 20 pounds per square inch after saturation by vacuum. Remolded subgrade specimens are allowed to moist cure and then are saturated by vacuum.

(3) Base course materials are formed at optimum moisture content in molds 5 inches in diameter by 14 inches high, are saturated by vacuum, and then tested triaxially under a lateral pressure of 20 pounds per square inch.

(4) The asphaltic surface course is assumed to have a minimum modulus of deformation of 15,000 pounds per square inch.

For a comparison of thicknesses required by the Kansas method of design the coefficients selected include:

(1) n equal 0.5 and m equal 1.0

(2) n equal 0.5 and m equal 1.5

(3) n equal to 1.0 and m equal to 1.0

(4) n equal to 1.0 and m equal to 1.5

A coefficient of n equal 0.5 and 1 correlate to rainfall amounts of 15.0 to 19.9 inches and 40.0 to 50.0 inches, respectively. A coefficient of m equal 1.0 and 1.5 relates to traffic densities of 1200 to 1500 vehicles per day and 4000 to 6000 vehicles per day, respectively (12).

Two assumed applications for the sand-asphalt and soil-cement will be considered in order to compare the suitability of the two stabilization methods. The first application is that in which the wearing surface and base course must be constructed on a subgrade which has less strength than the sand to be used in the base course. In this case the sand must be imported to the highway site. The second application is one in which the sand is the native material and actually constitutes the subgrade.

With a weaker subgrade than the sand it will be necessary to assume a subgrade modulus. The results of a triaxial test performed on a clay subgrade were extracted from reference (12) and are shown in Table 13. For this assumed subgrade the thickness of wearing surface is determined. Then with an arbitrary thickness of 2 inches of wearing surface the required depth of soil-cement or sand-asphalt is determined.

If the sand is the native material, the depth of wearing surface (C_p = 15, 000 psi) required above will be determined for the immersed series of specimens. For this case the average strength of the untreated sand will be assumed equal to the immersed strength of the sand-asphalt. From the triaxial results it was shown that the cohesion of the untreated sand was slightly increased but the friction angle was reduced by the addition of asphalt.

Results for the two assumed conditions are tabulated in Table 10 for the immersed sand-asphalt and the immersed soil-cement. The cured series have little meaning when analyzed by the Kansas method due to the moisture contents of only one per cent.

Sample computations of pavement thickness requirements, using the Kansas method are shown in Appendix A.

TABLE 10

THICKNESS OF SOIL-CEMENT AND SAND-ASPHALT BASE COURSES REQUIRED BY THE KANSAS METHOD

Test Series	n = 0.5 m = 1.0	n = 0.5 m = 1.5	n = 1.0 m = 1.0	n = 1.0 m = 1.5
Total thickness of pavement mat (C_p = 15,000 psi) required above assumed subgrade (C= 700 to 1700 psi) obtained from thickness charts				
	8.5	10.8	12.5	17.6
Thickness of base course material required above subgrade if pavement mat thickness (C_p = 15,000 psi) assumed to be 2 inches				
5% MC3 Immersed	8.9	12.1	14.4	21.4
3% MC3 Immersed	9.2	12.4	14.7	22.0
7% MC3 Immersed	9.3	12.6	15.1	22.4
Soil-Cement Immersed	4.0	5.5	6.5	9.7
Total thickness of pavement mat (C_p = 15,000 psi) required above untreated sand (C assumed equal 4400 psi) as native material obtained from thickness charts				
	2.0	2.0	2.6	6.8
Thickness of base course material required above native sand subgrade if pavement mat thickness (C_p = 15,000 psi) assumed to be 2 inches				
5% MC3 Immersed	From thickness charts (for light traffic), the required depths of sand-asphalt* and soil-cement** are zero or very near, thus the minimum thicknesses usually employed are recommended			7.7
3% MC3 Immersed				8.0
7% MC3 Immersed				9.0
Soil-Cement	"			5.0

* Empirical thickness of 6.0" from reference (4) --**Empirical thickness of 5.0" from reference (23) for an A3 soil

Texas Method

This method is outlined in references (15) (18) (19) and can be summarized as follows:

(1) Subgrade specimens are formed in molds six inches in diameter and eight inches high at optimum moisture content.

(2) The specimens are allowed to air dry overnight and then are oven-dried at 140° F. for eight hours.

(3) The samples are then permitted to stand and to absorb capillary moisture under a surcharge and lateral pressure both of one pound per square inch until equilibrium is reached.

(4) Each specimen is tested in compression at a constant lateral pressure. The six identical specimens are usually tested at lateral pressures of 0, 3, 5, 10, 15, and 20 pounds per square inch.

(5) From the principal stresses at the instant of failure, Mohr's diagram of stress is constructed.

(6) A portion of the Mohr envelope is transferred to a classification chart and the strength class of the material is determined to the nearest tenth.

(7) The depth of coverage in inches for a given wheel load is obtained by entering this data on a design chart.

This chart has been derived from experience and analysis of pavements, including service behavior, age, weight and volume of traffic, thicknesses of pavement layers, triaxial tests, soil

constants and gradation on subgrades, subbases and bases.

The thicknesses required for different class materials for a 9000 wheel load for design periods of ten years and twenty to thirty years were obtained from reference (18) and are shown in Table 14 of Appendix A.

In this method of analysis, triaxial results are performed on the native material and the required coverage of conventional flexible pavement is determined. With this method of analysis it is not possible to determine the depth of material which makes up the base course. Therefore, merely a comparison of the classes of the investigated materials will be attempted. (See Table 11)

The relationship of the Mohr's envelopes to the different classes is obtained by plotting the envelopes on the Texas Classification Chart which is illustrated in Figure 8.

TABLE 11

CLASSIFICATION OF SOIL-CEMENT AND SAND-ASPHALT BASE COURSES BY THE TEXAS METHOD

Test Series	Material Class	Material Suitability
5% MC3 Immersed	4.0	Poor Subgrade
3% MC3 Immersed	3.9	Poor Subgrade
7% MC3 Immersed	4.0	Poor Subgrade
Soil-Cement Immersed	1.0+	Good base course

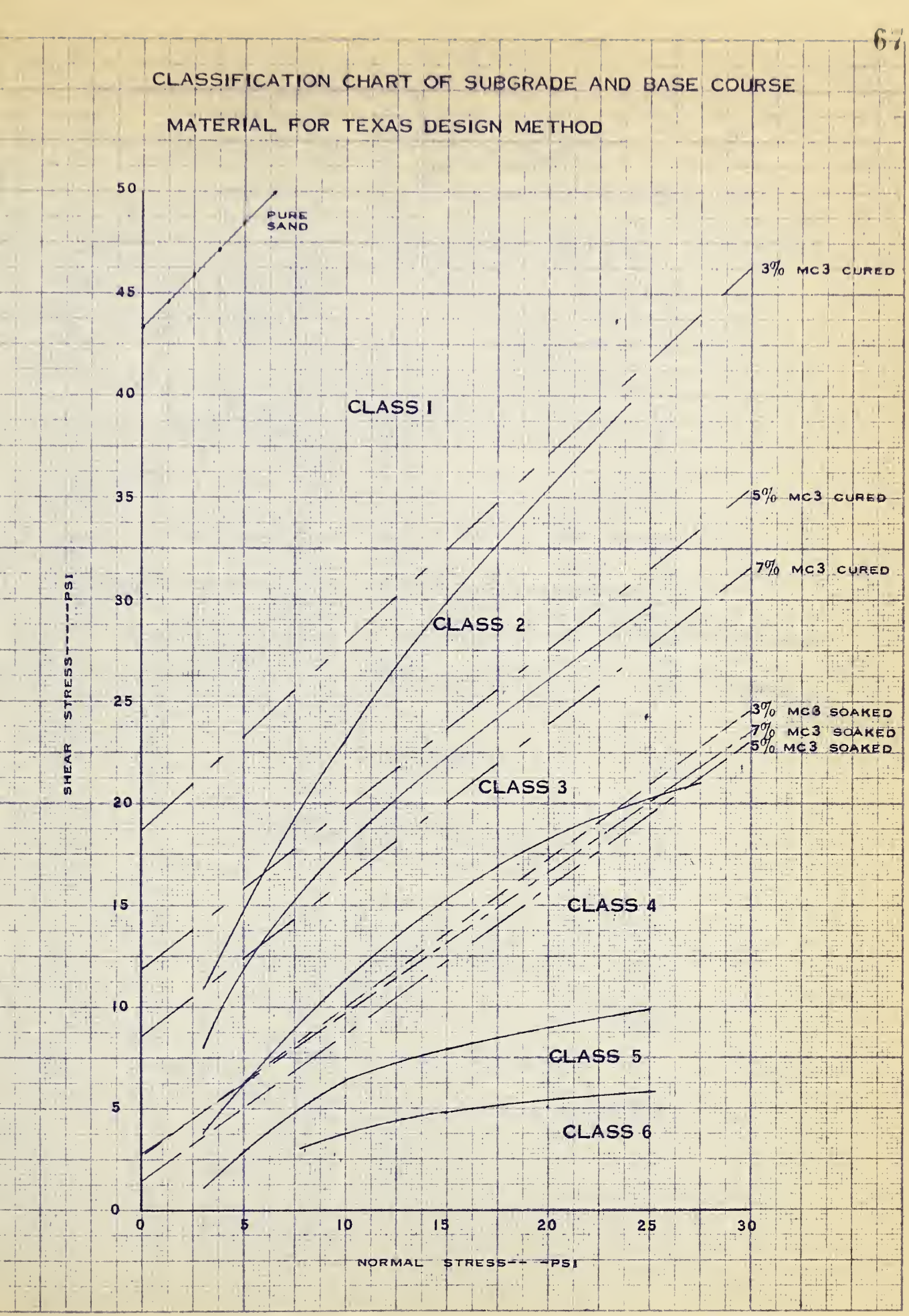

CLASSIFICATION CHART OF SUBGRADE AND BASE COURSE
MATERIAL FOR TEXAS DESIGN METHOD
PURE SAND
3% MC3 CURED
5% MC3 CURED
7% MC3 CURED
3% MC3 SOAKED
7% MC3 SOAKED
5% MC3 SOAKED
CLASS 1
CLASS 2
CLASS 3
CLASS 4
CLASS 5
CLASS 6
SHEAR STRESS----PSI
NORMAL STRESS---PSI
0
5
10
15
20
25
30
35
40
45
50

FIGURE 8

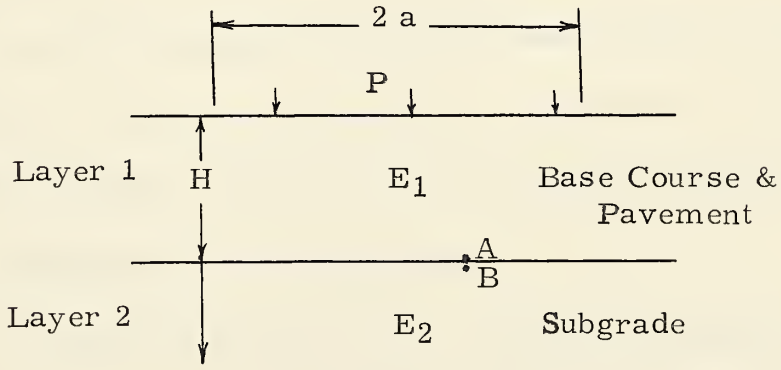
Application of Burmister's Theory of Two-Layered Systems to the Design of Pavement Thickness Using the Triaxial Test

From Burmister's theory of layered systems Hank and Scrivner (14) have developed a method for comparing the theoretical stresses for equilibrium at a contact surface with those stress conditions in the laboratory triaxial compression test. Table 3 of reference (14) gives influence values for the computed stresses, for the condition of perfect continuity at the interface, in a form convenient for use in the construction of Mohr's circles of stresses.

Points A and B of Figure 9 are the points where stresses were determined and Figure 9 also shows the meaning of the parameters entering into the stress equations. The stress equations have been reduced to a table form such as shown in reference (14).

FIGURE 9

PARAMETERS IN TWO-LAYER ANALYSIS



The meaning of the symbols in Figure 9 are:

E_1 = Young's modulus of upper layer

E_2 = Young's modulus of lower layer

a = radius of circular loaded area

H = depth of layer 1

R = a/H

P = vertical unit pressure acting on circular area at surface

The procedure for determination of thicknesses required is as follows:

(1) From stress-strain curves in compression determine the stiffness ratio E_2/E_1 for the materials proposed for use as base and subgrade. It is necessary to assume that the modulus in tension equals the modulus in compression for the upper layer.

(2) For a given wheel load determine the unit pressure, P, and the radius of contact area, a, to be used in the computations.

(3) If N and S represent the coordinates of a Mohr's envelope of rupture, then the corresponding coordinates for plotting are N/P and S/P. If the rupture envelope can be approximated by a straight line, with the equation;

$$S = C + N \tan O$$

where: C= cohesion

N = normal stress

0 = angle of internal friction

S = shear stress

then the line may be plotted on diagrams at an inclination of 0 degrees with the horizontal, and through the point C/P on the axis of zero normal stress.

For comparison of computed stress with laboratory measured strengths, the following loading constants were selected for convenience of interpretation:

$P = 100$ pounds per square inch

$a = 5.0$ inches

Wheel load = 7,854 pounds

The principle used is that computed stress circles which lie below the rupture line represent safe conditions of stress, while any circle which intersects the rupture line represents a state of stress which would cause failure at the interface.

For comparison of thicknesses required to prevent failure at the interface between the subgrade and the base course, the following conditions were arbitrarily selected:

(1) $E_2/E_1 = 0.1$

(2) $E_2 = 4000$ pounds per square inch with varying ratios of E_2/E_1.

(3) $E_2/E_1 = 0.3$

The results of applying the three above conditions to the soil-cement and sand-asphalt specimens are shown in tabular form in Table 12.

Sample calculations for this method of the two-layer system are shown in Appendix A.

TABLE 12

THICKNESSES OF MATERIALS REQUIRED ABOVE SUBGRADE

BY DESIGN METHOD OF TWO LAYER SYSTEM

Description of Material	E_1	$E_2/E_1 = 0.1$		$E_2 = 4000$psi		$E_2/E_1 = 0.3$	
		E_2	H	E_2/E_1	H	E_2	H
5% MC 3 Cured	17,600	1760	30	.23	23.5	5270	22
3% MC 3 Cured	23,200	2320	23.5	.17	21.0	6950	16.6
7% MC 3 Cured	11,700	1170	35	.34	24.0	3500	26.0
5% MC 3 Immersed	5,780	578	50+	.69	--	1730	50.0
3% MC 3 Immersed	5,400	540	50+	.74	--	1620	45.0
7% MC 3 Immersed	4,400	440	50+	.91	--	1320	45.0
Soil-Cement 7 Day Cure	50,250	5025	10.5	.08	11.5	15,100	6.0
Soil-Cement 7 Day Cure & 14 Days Immersion	62,000	6200	11.3	.06	12.5	18,600	7.2
Soil-Cement 28 Day Cure	71,700	7170	11.5	.055	12.8	21,500	7.2
Pure Sand 7 Day Cure	55,700	5570	15	.07	16.0	16,700	10.5

Discussion of Methods of Analysis

In order to apply the Kansas and Texas methods to the design of soil-cement and sand-asphalt base courses it must be assumed that both types of material act as a flexible material. There exists no nationally accepted and practiced method for structural design of pavements that consist of a soil-cement base course (23) but if the soil-cement is assumed to behave as a flexible pavement, then the flexible design methods may be applied. Recent investigations by the Road Research Laboratory in England (26) have shown that a soil-cement with a field compressive strength of 150 pounds per square inch or laboratory compressive strength of 250 pounds per square inch (due to superior curing conditions in the laboratory) behaves as a flexible base and not as a low-grade concrete. The United States practice (23) of flexible pavement design assumes that soil-cement displays characteristics of a flexible type pavement, or in some cases, of a semi-flexible type. Therefore, the flexible pavement design procedures were applied to both the sand-asphalt and the soil-cement. Because soil-cement possesses some rigidity it may be considered questionable whether the flexible pavement design is applicable to soil-cement.

Before discussing the reliability of the design thicknesses obtained, it may be helpful to point out that in Alberta, on a primary highway such as Highway 16, the soil-cement base course is comprised of six inches of soil-cement, two inches of road-mixed asphalt bound gravel and two inches of hot-mix asphalt pavement. Therefore, a

total thickness of ten inches is specified.

The variation in testing and sample preparation from that used in this work and that of the Kansas method are moisture content at testing, sample dimension, method of compaction and rate of load application. Probably, the most important variable is that of the moisture content at testing. The Kansas method requires that all specimens be saturated before testing; whereas, the soaked sand-asphalt specimens contained an average air-void content of thirty per cent. The rate of loading used in the Kansas test is 0.005 inches per minute; whereas, a rate of 0.01 inches per minute was employed in this investigation. Finally, the height to diameter ratio employed in this research was 2.32 compared to 2.80 for the Kansas specimens. The effect of these differences would generally tend to give strengths greater than would be obtained by adhering strictly to the Kansas specifications. The initial tangent moduli which were obtained by the best fit line were determined by personal judgment and may vary considerably and thus affect the results obtained in the Kansas method.

The Kansas design requires that for the severe traffic conditions of 4000 to 6000 vehicles per day and rainfall conditions of 40 to 50 inches per year, that thicknesses required above the assumed subgrade for the immersed sand-asphalt and soil-cement are 22.0 and 9.7 inches, respectively (See Table 10). For the less severe conditions of 1200 to 1600 vehicles per day and 15 to 20 inches of rainfall per year the thicknesses required for immersed sand-asphalt and

soil-cement are 9.0 and 4.0 inches, respectively (See Table 10). Consequently, there is a considerable range of thicknesses available by this method of design.

If the sand constitutes the subgrade, then the thicknessess of sand-asphalt and soil-cement required are much less than with a weak subgrade. Actually, the Kansas procedure indicates that for low traffic intensities and rainfall, the depth of material required above is equal to zero. A bituminous mat placed directly on an untreated sand results in poor bonding and movement of the mat. Consequently, the minimum thicknessess of sand-asphalt and soil-cement usually constructed are included in Table 10, where the Kansas method requires zero thickness.

From an analysis of the Kansas method, the sand-asphalt would not be attractive as a base course if material must be imported to the road site due to the excessive quantities of material which would require processing. For this case the soil-cement would be attractive as it would add structural strength to the pavement with thicknesses much less than the sand-asphalt.

From Table 10 it can be seen that the sand-asphalt is much more practical when the sand constitutes the native subgrade rather than when a weak subgrade is present. For this application the sand-asphalt may perform as well as the soil-cement because a structural thickness is not required above the sand.

The Texas method uses specimens very near saturation as they are allowed to absorb capillary water under a pressure of one pound per square inch. The specimen height to diameter ratio is 1.33 and a strain rate of 0.15 inches per minute is utilized. Thus, again the Texas method is using a sample which has much higher moisture content than the immersed specimens of this investigation and would have lower strengths than the specimens tested here. Conversely, due to the low height to diameter ratio and the rapid strain rate of the Texas method the specimens would tend to be stronger than the immersed specimens of this investigation. Consequently, it can be concluded that the specimens tested in this investigation would more nearly approximate the strengths obtained in the Texas design method rather than those of the Kansas method.

The classes of the soil-cement and sand-asphalt by the Texas method indicate that the soil-cement is the superior base course material (See Table 11). Using this method it is not possible to determine the thicknessess of the materials investigated.

Both the Texas and Kansas methods of design attempt to test all materials at the same moisture condition. This procedure may be wise but could result in very uneconomical designs. For example, in this investigation after fourteen days immersion the sand-asphalt specimens absorbed much more water than the immersed soil-cement specimens. Examination of results show that the immersed soil-cement specimens actually gained strength under the beneficial curing conditions.

of soaking. Thus immersion is not a severe test for this soil-cement. Again, the problem arises of duplication of field conditions in the laboratory.

The design method introduced by Hank and Scrivner and based on Burmister's theory of layered systems is a theoretical method which evaluates a material under the testing conditions and thus it is possible to compare materials under different moisture conditions. This two-layer theory as applied to pavement and base course appears to be appropriate for highway design. It is now necessary to correlate the theoretical analysis with experimental work before a completely rational design procedure for flexible pavements can be accepted.

Because Burmister's stress equations are formulated such that the tension stresses are obtained, it is thus necessary to obtain the tensile strength of the material. Because it is not possible to obtain the tensile strength of some base course materials the rupture envelope is extrapolated to the tensile side of the Mohr's diagram.

The method of Hank and Scrivner gives thicknesses which are dependent on the strength of the subgrade as well as the base course material. Using a subgrade modulus of 4000 pounds per square inch results in a design thickness for the seven day cured soil-cement of 11.5 inches which is near that now being used by the Department of Highways in Alberta. For the same subgrade, the average depth of the cured sand-asphalt required was 22.8 inches. For a subgrade modulus of 4000 pounds per square inch it was not possible to extra-

polate the Burmister tables for the immersed specimens, but for a subgrade modulus of near 1600 pounds per square inch a thickness of 40 inches of the immersed sand-asphalt is required. Consequently, the cured sand-asphalt requires twice the depth as the soil-cement and the immersed sand-asphalt requires four times that of the soil-cement. From the method based on Burmister's theory it can be seen that the thicknesses required by the immersed sand-asphalt would make a base course very uneconomical. One of the shortcomings of this method is that traffic volume is not accounted for, thus there is no difference in thicknessess for varying traffic intensities.

The thicknessess of sand-asphalt obtained by the two-layer method of analysis are too great to give an economical design. The soil-cement is rated as the most practical material on the basis of the thicknessess obtained by the two-layer method. Of course, this method has not been correlated with field conditions and these depths may result in over-design.

From the results obtained from this investigation and analyzed by the three methods, it is indicated that the soil-cement is the structurally superior base course material. The sand-asphalt may be competitive with the soil-cement when the sand is the native material and sufficient bearing capacity is available in the untreated sand.

In addition to the three methods of analysis employed in this investigation, a method has been formulated by N. W. McLeod (25) in which the ultimate strength of a highway structure is considered. The

method involves calculating the ultimate strength of a flexible pavement on the basis of the C and ϕ values for each layer and assuming a logarithmic spiral failure curve. The C and ϕ values are obtained from the laboratory triaxial test. In essence, this method involves the determination of C and ϕ values for an equivalent homogeneous material having the same ultimate strength as the layered system of the flexible pavement. The critical circular arc is found by trial and error and is the arc along which the shearing resistance of the soil will support the lowest ultimate load. The location of the failure surface is very laborious to determine manually but with the aid of a computer program the task would be greatly simplified. This procedure is therefore another method of evaluating materials for which triaxial results are available.

CHAPTER VIII

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

From the laboratory investigation and the methods of analysis, the following conclusions result:

(1) The maximum unconfined compressive strength of a sand-asphalt stabilized mixture is obtained at a molding volatile content near that required for optimum density. But strength is not greatly affected by changes in density.

(2) The cured strength of a sand-asphalt mixture is reduced with the addition of increasing percentages of asphalt. This phenomena probably is due to the weaker asphalt-soil bonds replacing the stronger soil to soil surface tension forces.

(3) The major role of asphalt in stabilizing of this sand is to give the sand cohesion during immersion. The cohesion upon immersion is greatly decreased from the cured specimens but tends to reach a minimum value of 2 pounds per square inch regardless of the asphalt content.

(4) The soil-cement has a higher shear strength than the sand-asphalt. From the methods of analysis it is shown that the sand-asphalt may be competitive if the sand is the native material and only requires a stabilized surface rather than a structural thickness.

(5) Due to the low permeability of the soil-cement, the absorption of water upon immersion is low and only tends to facilitate hydration and increase the strength. Consequently, immersion is not a limiting condition for a soil-cement which has cured for seven days.

(6) The triaxial compression test is superior to the unconfined compression test when comparing the soil-cement and sand-asphalt which have very different strength characteristics.

Recommendations for Future Research

(1) In order to obtain a greater understanding of the mechanics of asphalt-stabilization it would be advantangeous to investigate soils with different grading and textural characteristics. Observations by others have shown many varying results which mainly tend to be due to the variations in soils (24). It is suggested that the unconfined test be utilized if such variables as density and asphalt content are to be investigated. After the basic trends have been established with the different soils, the triaxial test may be used to compare the optimum conditions for the different soils.

(2) The duplication of specimens in the triaxial compression test appears to be unnecessary. Each Mohr's circle tends to determine the accuracy of all other circles. By testing specimens at six different lateral pressures a well-defined Mohr envelope could be obtained and the required number of specimens would be halved.

(3) The stabilization of sand may be performed very satisfactorily by using a cementing agent and waterproofing agent both in the same mix. The combination of the two additives may not be structurally superior to the soil-cement of this investigation but perhaps soils with greater amounts of fines could be greatly improved by the use of both additives (23).

(4) The one application of load at a slow rate in the laboratory compared to the many repetitions of rapid loading in the field may show different strength characteristics for the soil-cement and the sand-asphalt. Thus to more nearly simulate loading conditions of the field, repetitive loading under triaxial conditions should be valuable.

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(14) Hank R. J. and Scrivner F. H. "Some numerical Solutions of Stresses in Two and Three-Layered Systems". Proceedings of Highway Research Board -Volume 28, 1948

(15)"Thickness of Flexible Pavements" Current Road Problems-Revised Edition-No-8-R-Highway Research Board -1949

(16) State Highway Commission of Kansas Research Department-Flexible Pavement Thickness Charts for High Volumes of Traffic-State Highway Commission of Kansas - 1953

(17) Lacey D. L. -,"Flexible Pavement Design Correlated with Road Performance" Highway Research Board Research Report 16B-1954

(18) McDowell C. - "Triaxial Tests in Analysis of Flexible Pavements" Highway Research Board Research Report 16B--1954

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(20) McLeod N. W. "Fundamental Principles of Mechanical Stabilization" Proceedings of the Association of Asphalt Paving Technologists--Volume 14, 1943

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(26) MacLean D. J.- "The Application of Soil Mechanics to Roads and Engineering Foundations". Journal of Institute of Municipal Engineers.-1955

ANNOTATED BIBLIOGRAPHY

The references in this bibliography are articles related to soils stabilized with asphalt. There are no references to asphalt specifications or asphalt production. Each article on soil-asphalt stabilization has a reference number assigned to it and a short annotated summary of its contents. The reference number is composed of two parts (35-1). The first part of the reference number indicates the year of publication 1935, and the second part refers to the arrangement within that particular year. The references are arranged chronologically according to the earliest year of publication and alphabetically according to authors within the particular year. Where the name of the author was not given, the article is referenced last in that year.

The references in this bibliography were those which were available in the Rutherford library and the Civil Engineering library of the University of Alberta. The articles were obtained mainly from publications of the Highway Research Board and the Association of Asphalt Paving Technologists.

35-1 McKesson C. L. "Soil Stabilization with Emulsified Asphalt" Proceedings Highway Research Board Volume 15-1935

Author believes that emulsified asphalts are only means that will give satisfactory results on soils containing a large percentage of material smaller than .005 mm. The quantity of emulsified asphalt required increases as particle size decreases. It is believed that absorption tests should be of the capillary type as road grades are subject to capillary water action mainly. It is realized that a very stable emulsifying agent is necessary to prevent breakdown before mixing is completed.

36-1 Reagel F. V. and Schappler R. C. "Use of Cutbacks and Road Oils in Soil Stabilization". Proceedings of Highway Research Board Volume 16, 1936--Page 359

This article stresses the need for a cheap road base which is waterproof. The author states that by adding bituminous material to a soil it is possible to maintain the moisture content below the plastic limit or raise the plastic limit consequently, increasing the systems stability.

Two methods are given for field construction; one in which the subgrade is scarified to 5 inches followed by the introduction of bituminous material and then compaction; the other method is scarifying, and windrowing the material and then mixing asphalt with a travelling mix plant.

37-1 Weathers H. C. "Sand-Bituminous Stabilization". Proceedings of Highway Research Board Volume 17, 1937--Page 521

This paper describes some of the first approaches to sand-bituminous stabilization in Florida. Up until this time the methods used were trail and error. A stability test is recommended in which the apparatus used is a lever arm system, requiring a minimum stability of 25 psi. Specifications are given for cutback asphalt and coaltar. Construction procedures used on 314 miles of sandy soil are described.

38-1 Bratt A. V. "Asphalt Emulsion Stabilization on Cape Cod". Proceedings of Highway Research Board Volume 18, 1938 Page 289

This short article concerns the 75 miles of sandy soil on Cape Cod which contains no coarse aggregate. It was found that by using emulsion stabilization, a base as stable as using imported aggregate could be obtained and half the cost was involved. A brief description of construction procedures is given.

38-2 Martin G. E. " Soil Stabilization with Tar". Proceedings of Highway Research Board Volume 18, 1938--Page 275

The author outlines the laboratory procedure for designing soil-tar mixtures based on Hubbard-Field stability with immersion and capillary action. It is stressed that it maybe more economical to add aggregate to a very fine soil than use a large percentage of tar. The construction procedure used

includes a number of stages including; aggregate admixture, loosening, pulverizing, water application, tar application, mixing, compaction and tack coat.

38-3 McKesson C. L. "Bituminous Emulsion Stabilized Roads". Proceedings of Highway Research Board Volume 18, 1938

Author discusses generally the design of emulsified-soil mixtures and properties of emulsified asphalt. The field construction procedure used at this time is explained.

38-4 Muir L., Hughes W., and Browning G. "Bituminous Stabilization Practices in the United States". Proceedings of Highway Research Board Volume 18, 1938

This article describes the work done by the different states using asphalt-stabilization. In Missiouri considerable mileage was stabilized and it was found that cutbacks and road oils performed as well as emulsions. The largest portions of asphalt-stabilization carried on was in Florida and California. Work was also completed in South Carolina, North Carolina, Kentucky, Ohio, Iowa, Nebraska, and Wyoming.

38-5 Reagel F. V. "Asphaltic Binder Stabilized Roads". Proceedings of Highway Research Board Volume 18, 1938

This article describes three field procedures used in Missiouri and neighbouring states for the asphalt-stabilization of soils. The methods include; the machine mix procedure in which material is windrowed and then mixed in a pugmill, the second is that of suboiling in which the scarifying blades are also nozzles which shoot asphalt into the soil, and thirdly is the road mix type in which the mixture is turned and disced as the asphalt is applied.

38-6 Roediger J. C. and Klinger E. W. "Soil Stabilization Using Asphalt Cutbacks". Proceedings of Highway Research Board Volume 18, 1938

This article gives the design procedure based on Hubbard-Field stability with uncured partially immersed specimens. The field procedure described is that of the road mix type of operation.

39-1 Loughborough T. F., Miller A. M., Schafer N. F. and Telly A. C. "Use of Tar in Base Stabilization". Proceedings of Highway Research Board Volume 19, 1939

The authors give a brief outline of the design on which field work is based and a very detailed description is given of the construction procedures used in the authors home states of Virginia, Indiana and Nebraska. It is generally agreed that manipulation costs will be increased materially if the moisture content is too high or too low. Low moisture content requires excess working to get coating, whereas, high moisture contents require additional manipulation to permit escape of moisture.

39-2 Roedinger J. C. and Klinger E. W. "Soil Stabilization Using Asphalt Cutbacks". Proceedings of the Association of Asphalt paving Technologists Volume 10, 1939

This article gives a detailed procedure for laboratory design of cutback-soil mixtures. It is recommended that uncured specimens be tested so that the worst condition is known and the water absorption test is carried out under partial immersion conditions so that air-blockage is prevented and consequently equilibrium conditions are reached faster. Design is based on Hubbard-Field tests on uncured specimens before and after water immersion but water absorption and swell must not be excessive. Results and field procedures are given for three test strips using MC2 with different soil types.

39-3 Scoggin B. I. "Soil Stabilization with Emulsified Asphalt in the Mid-Continent Area". Proceedings of Highway Research Board Volume 19, 1939

This review lists emulsified asphalt-soil stabilization projects in Texas, Oklahoma, Iowa and Nebraska, with total area equivalent to 250 miles of 18 ft. roadway. Maximum dehydration is a more important factor effecting durability of soil stabilized by use of emulsified asphalt, than maximum density at time of compaction. For successful stabilization by use of emulsified asphalt, design the base with a 4 inch minimum thickness and with sufficient emulsion to give a finished base meeting some known efficiency requirement. To provide for minimum moisture content before compaction is the best method and depend upon subsequent traffic to increase density rather than upon weather conditions favorable to complete drying of 4 to 6 inch depth.

39-4 Thurston R. R. and Weetman B. "The Stabilization of Soils with Emulsified Asphalt". Proceedings of the Association of Asphalt Paving Technologists Volume 10, 1939

This article describes tests used to determine emulsibility of the asphalt and its suitability for stabilization of different soils. Tests described include; lever type compression testing apparatus, calcium chloride demulsibility test, stone coating test, cement mixing test, and dehydration test. A general procedure for field application is given recommending that the soil and asphalt be mixed at the liquid limit of the soil.

40-1 Cape E. B. "Test Methods Used in the Design and Control of Soil-Bituminous Mixtures in Texas". Proceedings of the Association of Asphalt Paving Technologists Volume 12, 1940

A modified Bearing Value test is examined for use in design of asphalt mixes. It is indicated that water content is important in compaction. The procedure and main factors are discussed in the construction of 195 miles of soil-bitumen bases in Texas.

40-2 McKesson C. L. "Recent Developments in the Design and Construction of Soil-Emulsion Road Mixtures". Proceedings of Highway Research Board Volume 20, 1940

This paper reviews the theory of emulsified asphalt-stabilization. Improvements in field operations are discussed including; mixing soil and asphalt, proportions of soil and emulsified asphalt, rolling, drying, and stabilization of sand with emulsified asphalt. It was found that drying in 2 inch layers resulted in a prepared 6 inch base much quicker than if dried in 6 inch layers.

41-1 Gardner H. "The Use of Bitumen in Soil Stabilization". Institue of Petroleum Journal Volume 27, 1941

Author states that there is an optimum water content for the most stable design. The action of bitumen stabilization is discussed and a penetration test is given for determining the optimum mix. An absorption test is suggested and minimum requirements for a soil-bitumen mix given. Data is presented from a field test using 3% oil with 60% clayey silt with an addition of 40% pit sand.

41-2 Markwick A. H. D. "Soil Classification and its Bearing on Soil Stabilization". Institute of Petroleum Journal Volume 27, 1941

This article explains the importance of classification tests and grading analysis for determining the most appropriate stabilizing agent or method. Economic stabilization depends on the character of the soil. A detrimental soil gradation for stabilizing may be improved by addition of particles which the soil lacks. This method of improved gradation maybe more economical than stabilization with asphalt bitumen, which at this time was the most popular stabilizer excluding mechanical stabilization.

41-3 McKesson C. L. and Mohr A. W. "Soil-Emulsified Asphalt and Sand-Emulsified Asphalt Pavement". Proceedings of Highway Research Board Volume 21, 1941

This article relates mainly to sand-emulsified stabilization. A modified Florida bearing value test is suggested to test untreated and treated mixtures of sand. It is stressed that usual absorption and stability test methods on soil-emulsified asphalt mixtures should not be used for designing asphalt-sand mixes. Sands of low stability can be made suitable for use by the addition of fine non-cohesive fillers.

42-1 Benson J. R. and Becker C. J. "Exploratory Research in Bituminous Soil Stabilization". Proceedings of the Association of Asphalt Paving Techologists Volume 14, 1943

The authors introduce the "fluff point" as being the moisture content at which asphalt should be mixed with soil. Also intimate mix is proven to be detrimental because of spreading the asphalt in too thin a layer. Also reports on cement and lime additions to soil-asphalt systems. Cement was found to be most applicable. Soil types from sands to clays were investigated.

42-2 Endersby V. A. "Fundamental Research in Bituminous Soil Stabilization". Proceedings of Highway Research Board Volume 22, 1942

This article gives a complete discussion of the theory of asphalt-soil stabilization and the action of the asphalt in the soil. The theories discussed include the "plug theory" and the "intimate mix theory". Author stresses that oil is not used to produce cohesion, but to protect the existing hydraulic cohesion (especially in fine-grained soils).

Phase mixing is explained from publication by Benson and Becker. The "fluff point" for mixing moisture content is suggested as being the best condition to obtain more nearly a intimate mix. Author also believes that absorption tests should be more severe than capillary absorption.

43-1 Holmos A. and Klinger E. W. "Field Procedure for the Design of Cutback Asphalt-Soil Mixtures". Proceedings of Association of Asphalt Paving Techologists Volume 14, 1943

The article describes a quick method for field classification of soils in relation to amount of gravel (retained on #10 sieve), sand (minus #10 and retained on #200) and fine soil (minus #200). The design of mixes is based on the Hubbard-Field apparatus and a minimum load of 1200 pounds is recommended for an uncured specimen after molding. If the specimen has stability greater than 1200 pounds a water absorption test is run and stability must be a minimum of 400 or 500 pounds depending on which of the two given absorption tests are used. Also volumetric swell must be less than 5%.

43-2 McKesson C. L. "Soil Stabilization with Emulsified Asphalt". Proceedings of Association of Asphalt Technologists Volume 14, 1943

The author explains many properties of emulsified asphalt mixes and their importance in the highway field. Asphalt is used primarily to maintain the water content of a soil below that for maximum compaction. Soils containing 30% of material passing the #200 sieve maybe stabilized using emulsified asphalt but if a greater percentage passes it will only be economical to add coarser aggregate. The design procedure suggested is to prepare specimens at different asphalt contents and high moisture contents, the loose mixture is cured and compacted at optimum moisture, specimens are cured and then soaked by capillary absorption, finally being tested by one of the many stability tests. A outline for field construction of soil-asphalt bases and sand-asphalt bases is given.

44-1 The Soil Stabilization Panel of the Standardization Sub-Committee No. 7- Asphaltic Bitumen "Report on tests for Soil Stabilization" Institute of Petroleum Journal Volume 30, 1944

This report is a first account of tests suggested and developed by the committee for investigations of bitumen

stabilization. Methods given include; preparation and mixing of soil bitumen, determination of moisture content (including volatiles), cone penetration test for stability, unconfined compression test for stability, capillary water absorption test, determination of bulk density of soil, and water absorption test by immersion.

47-1 The Soil Stabilization Panel of the Standardization Sub-Committee No. 7- Asphaltic Bitumen "Second Report on Tests for Soil Stabilization". Institute of Petroleum Journal Volume 33, 1947

This article gives the work done since 1944 by the Committee. Test methods are given which include; stabilizer content of soils containing bituminous stabilizers by the cold extraction method and hot extraction methods, and water resistance of stabilized soil base. Three methods are given for determining the field density of compacted samples. At this time the Committee suggested that a freeze-thaw test be developed to simulate field conditions.

49-1 Andrews A. C. and Noble G. G. "A study of Bituminous Soil Stabilization by Methods of Physical Chemistry". Proceedings of Highway Research Board Volume 29, 1949

The authors explain the action of asphalt on soils from plastic clays to fine sand in relation to physical chemistry. Studies included; ability of asphalts to imbibe and retain moisture, contact angle characteristics, surface pressures and surface charges (electrophoresis studies). These physical chemical characteristics were analyzed for the different soils and different conditions. Effects of 85 admixtures were studied and the amines were found to be the most promising.

49-2 Rice J. M. and Goetz W. H. "Suitability of Indiana Dune, Lake and Waste Sands for Bituminous Pavements". Proceedings of the Asphalt Technologists Volume 18, 1949

This paper investigates several sands native to the state of Indiana which have a uniform particle size. It was found that sands with smooth surface texture, lack of natural fine material, and one-sized gradation are inferior to sands containing the opposite ingredients in regard to compressive

strength when mixed with RC2 and limestone filler. Sands containing large amounts of clay have high dry strengths but low water resistance. By increasing the asphalt content, the water resistance can be increased. Moisture contents in excess of 2% in the sand when mixed with RC 2 produce poor results.

52-1 Turnbull W. J. and Foster C. R. "Bituminous Stabilization". Proceedings on Soil Stabilization--MIT--June 1952

This article discusses the processes of bitumen soil stabilization. It is emphasized that emulsions and cut-back soil mixtures must be dried before maximum stability is reached. Stabilization of sands can be very economical but fine-grained soils cannot usually be stabilized economically. This short paper is concluded by discussion of stabilization by experts who generally review the progress made in bituminous-soil stabilization to date.

56-1 Michaels A. S. and Puzinauskas V. "Additives as Aids to Asphalt Stabilization of Fine-Grained Soils". Highway Research Board Bulletin 129--1956

The object of this investigation was to examine the effect of certain additives on the stabilization of a clayey silt with asphalt. The most recent theory of cutback and emulsion stabilization is outlined. From the action of the two stabilizing asphalts, the cutbacks appear to give the best waterproofing properties, but it is attempted to improve both with chemical additives. Antistripping additives such as lauryl amine are used which reduce the tendency for asphalt to "strip" from the soil upon wetting.

The most beneficial reactive chemical was found to be P_2O_5.

58-1 Csanyi L. H. and Nady R. M. "Use of Foamed Asphalt in Soil Stabilization". Proceedings of Highway Research Board Volume 37, 1958

This article is a description of design and field procedures of a foamed-asphalt stabilization project in Iowa. Asphalt cement in the form of a foam can coat fine-grained soil particles in a cold, damp condition. The test results indicate a desirable asphalt content of 6 per cent. To secure a uniform

distribution of foamed asphalt with single coatings on individual soil particles, the desired quantity of asphalt should be introduced during a single processing pass.

58-2 Herrin M. "Drying Phase of Soil-Asphalt Construction". Highway Research Board Bulletin 204----1958

The results of this study indicate that soils stabilized with cutbacks need to be dried out before compaction to provide high initial strength. Although drying is needed before the mixture is compacted, too dry a mixture is not desirable. The drier the mixture is at the time of compaction, the greater the air void content is; thus the greater is water absorption. It is indicated by this study that a cutback stabilized material cannot be compacted to a density requirement, that is, high strength is not related to high weight.

58-3 Michaels A. S. and Puzinauskas V. "Improvement of Asphalt-Stabilized Fine Grained Soils with Chemical Additives". Highway Research Board Bulletin 204---1958

It was found that P_2O_5 is more effective in fine-grained soils than in coarser soils. Soils treated with $P_2O_5$and/or amines have high strengths if cured and immersed but there is no strength gain over untreated mixes if immersed after compaction. This article besides giving results of different chemical additives, postulates from the results the actual physical changes and properties that occur in a asphalt-soil mix.

59-1 Katti R. K., Davidson D. T., and Sheeler J. B. "Water in Cutback Asphalt Stabilization of Soil". Iowa Experiment Station-- Highway Research Board Bulletin 241 --1959

Compaction of a soil-cutback asphalt-water system immediately following mixing produces a more stable product than a procedure in which a drying back period is included between mixing and compaction. The percentage of mixing water required to produce maximum stability, maximum density, minimum total moisture content after immersion and minimum expansion is different for each property mentioned. A compromise moisture content (CMC) for mixing may be found at which the variance from the best value of the properties will be a minimum. The CMC is very close to the mixing moisture content at which maximum moisture content and at which maximum standard Proctor density of the soil-

cutback asphalt-water system occurs.

61-1 Kallas B. F. and Puzinauskas V. "Stabilization of Fine-Grained Soils with Cutback Asphalt and Secondary Additives". (Presented January 1961 Highway Research Board)

Effects of asphalt on density and strength of compacted mixtures appear to depend on the gradation characteristics of soil. Waterproofing effects are improved by the use of more asphalt. Of four testing methods used, all appeared to give the same relative results, therefore it is suggested to use the most convenient method.

Chemical additives are discussed and there effect on the properties of a asphalt-soil mixture explained.

APPENDIX A

Sample Calculations Using the Kansas Method

Sample Calculations Using the Two-Layer System

Sample Calculation for Analytical Solution and Confidence Limits for Mohr Envelopes.

Sample Calculations of Void Properties of Immersed Sand-Asphalt.

APPENDIX A

Pavement Thickness Calculations Using the Kansas Method

The thickness computation for the 5 per cent MC3 immersed for 14 days will be used as an example. The rainfall coefficient and traffic density coefficient selected were 0.5 and 1.0, respectively. With these two coefficients the next step was to select the appropriate chart which was derived by inserting the coefficients in the design formula. The charts are shown in Figures 10 and 11 and were obtained from reference (16). The coefficients selected for comparison of thicknesses were chosen merely to give a range of design thicknesses and do not necessarily correlate with any particular region of Alberta.

With triaxial compression results from a subgrade on which a base course and pavement are to be placed, arbitrarily selected stress-difference values and calculated moduli of deformation are entered into the chart. The subgrade assumed was that of a clay for which triaxial results are given in reference (12) and shown below in Table 13.

TABLE 13

TRIAXIAL RESULTS ON CLAY SUBGRADE

Stress Difference	Strain	Modulus (psi)
3	0.00175	1710
4	0.00250	1600
5	0.00350	1430
6	0.00600	1000
7	0.01000	700

It is necessary to choose a deviator stress or stress difference from Table 13 along with a calculated modulus of deformation and then plotting this point on the design chart. Another value of deviator stress is obtained but selected so that is plots on the opposite side of the design line as the previously chosen coordinate. The two points are joined and their intersection with the design curve gives the thickness of mat required. For this example deviator stresses of 6 and 7 pounds per square inch and moduli of 1000 and 700 pounds per square inch are chosen, respectively (See Figure 10). The plotting of these two points on Figure 10 gave a thickness of pavement of 8.5 inches above the subgrade. To obtain the desired thickness of 5 per cent MC3 sand-asphalt required with a pavement thickness of two inches and modulus of 15, 000 pounds per square inch the design formula;

$$t_t = (T - t_p) \sqrt[3]{C_p / C_t}$$

is utilized and for this example the symbols have the following values:

T = 8.5 inches
t_p = 2.0 inches
C_p = 15, 000 psi
C_t = 5, 780 psi (obtained from stress-strain curve for 20 psi lateral pressure shown in Figure 6).

Thus by substituting the above values in the design formula a required thickness of sand-asphalt of 8.9 inches is found. The total thickness requirement is 10.9 inches including the 2 inches of asphaltic pavement.

PAVEMENT THICKNESS CHART FOR KANSAS DESIGN METHOD

FOR N = 0.5 S = 0.1 INCHES

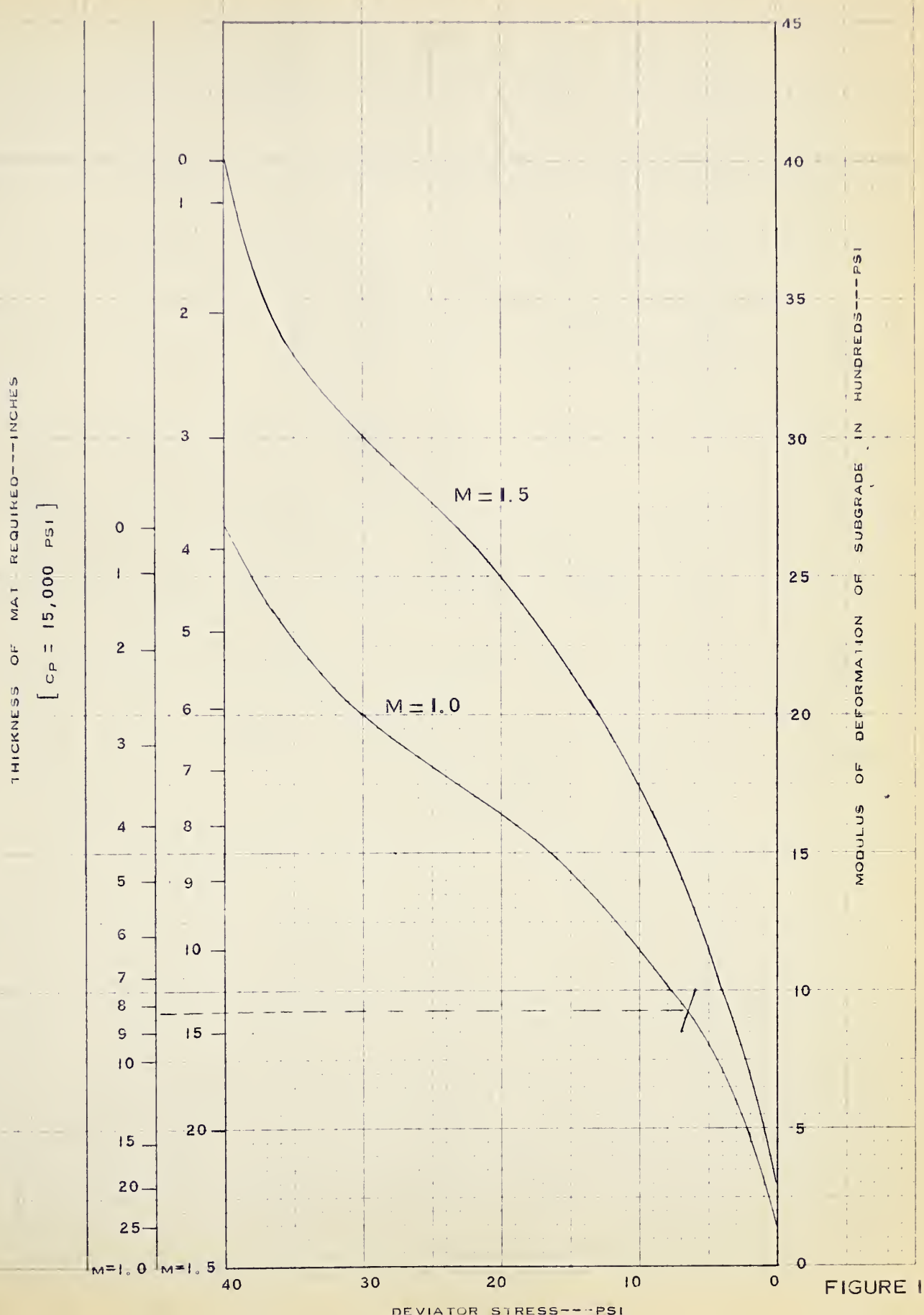


FIGURE 10

PAVEMENT THICKNESS CHART FOR KANSAS DESIGN METHOD

FOR N = 1.0 B = 0.1

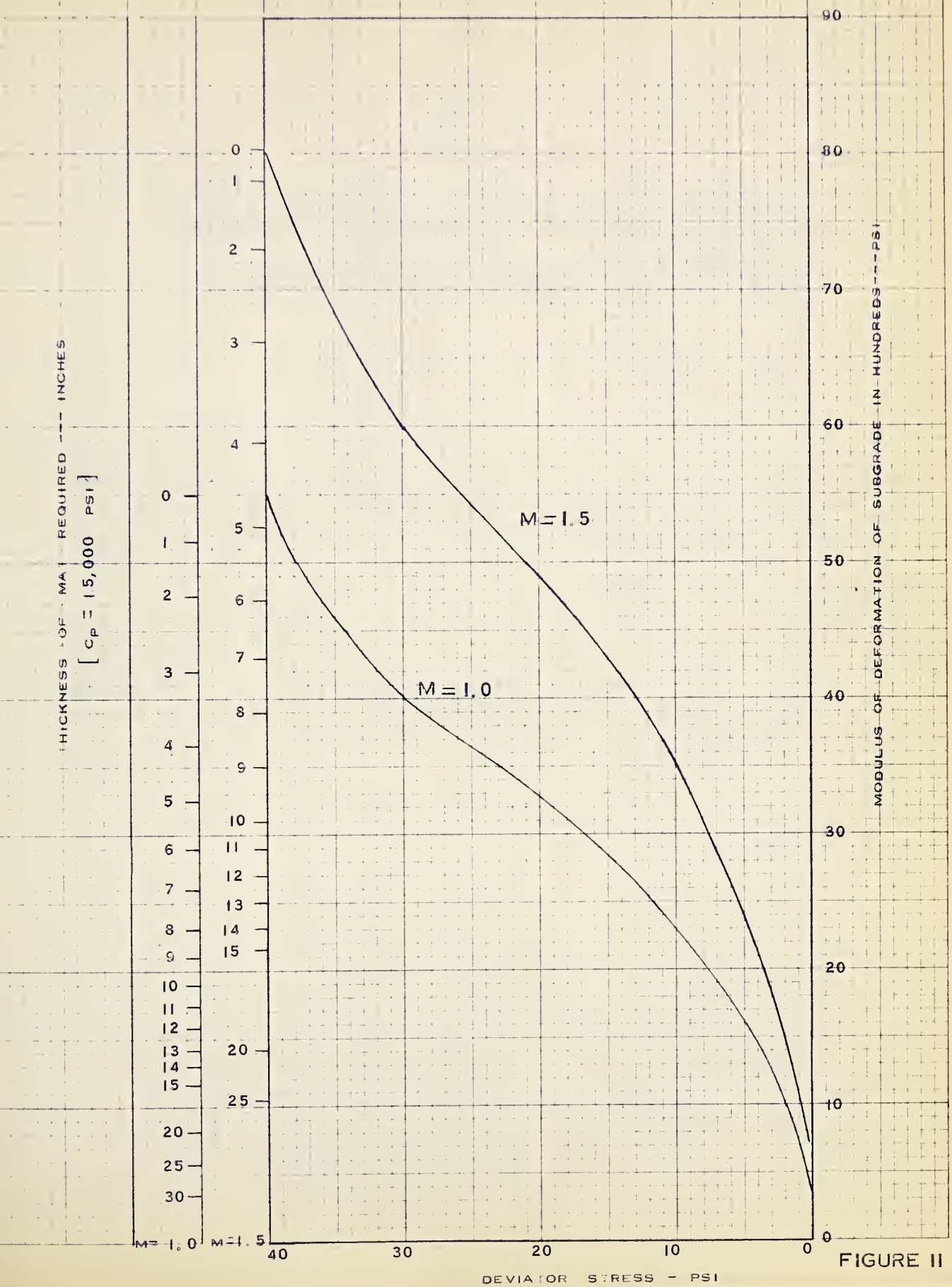


FIGURE 11

TABLE 14

THICKNESSES OF DIFFERENT CLASS MATERIALS FOR 9000 POUND WHEEL LOAD BY TEXAS METHOD

Class	1	2	3	4	5	6
Depth of Cover Required for 10 year design	0-1	1-2.5	2.5-7.5	7.5-12.0	12-16.5	16.5-
Depth of Cover Required for 20 to 30 year design	0-1	1-3.2	3.2-10.6	10.6-17	17-22.3	22.3-

Pavement Thickness Calculations Using the Method of Two-Layered Systems

For an example illustration of this method of design the soil-cement series cured for seven days will be selected. A subgrade modulus of deformation of 4000 pounds per square inch will be selected thus giving a modular ratio of 0.08 . The soil-cement had a modulus of deformation of 50,250 pounds per square inch at a lateral pressure of 20 pounds per square inch. With the modular ratio of 0.08 it is necessary to obtain centers and radii of Mohr's circles for various values of H/a. This data is obtained from tables in reference (14). The values of H/a chosen were 2.0 and 2.5 as it was found by trial and error that the rupture line for the soil-cement plotted between the two circles. From reference (14) by interpolation of tables the following data was obtained for $E_2/E_1 = 0.08$:

H/a	Center of Stress Circle	Radius of Stress Circle
2.0	.322	-.409
2.5	.222	-.283

The two circles are plotted in Figure 12. The Mohr's rupture envelope is also plotted and extended through to the tension side of the shear ordinate. Now it is merely a problem of choosing a circle which is tangent to the rupture line, thus giving the equilibrium conditions for perfect continuity at the interface between the subgrade and soil-cement. By trial and error the Mohr's circle

of $H/a = 2.3$ was found to be near tangent to the rupture line. Therefore, the thickness of material required equals 2.3 multiplied by the loaded radius of 5 inches. This results in a required thickness of 11.5 inches.

THICKNESS OF PAVEMENT DETERMINATION USING BURMISTER'S THEORY OF LAYERED SYSTEMS

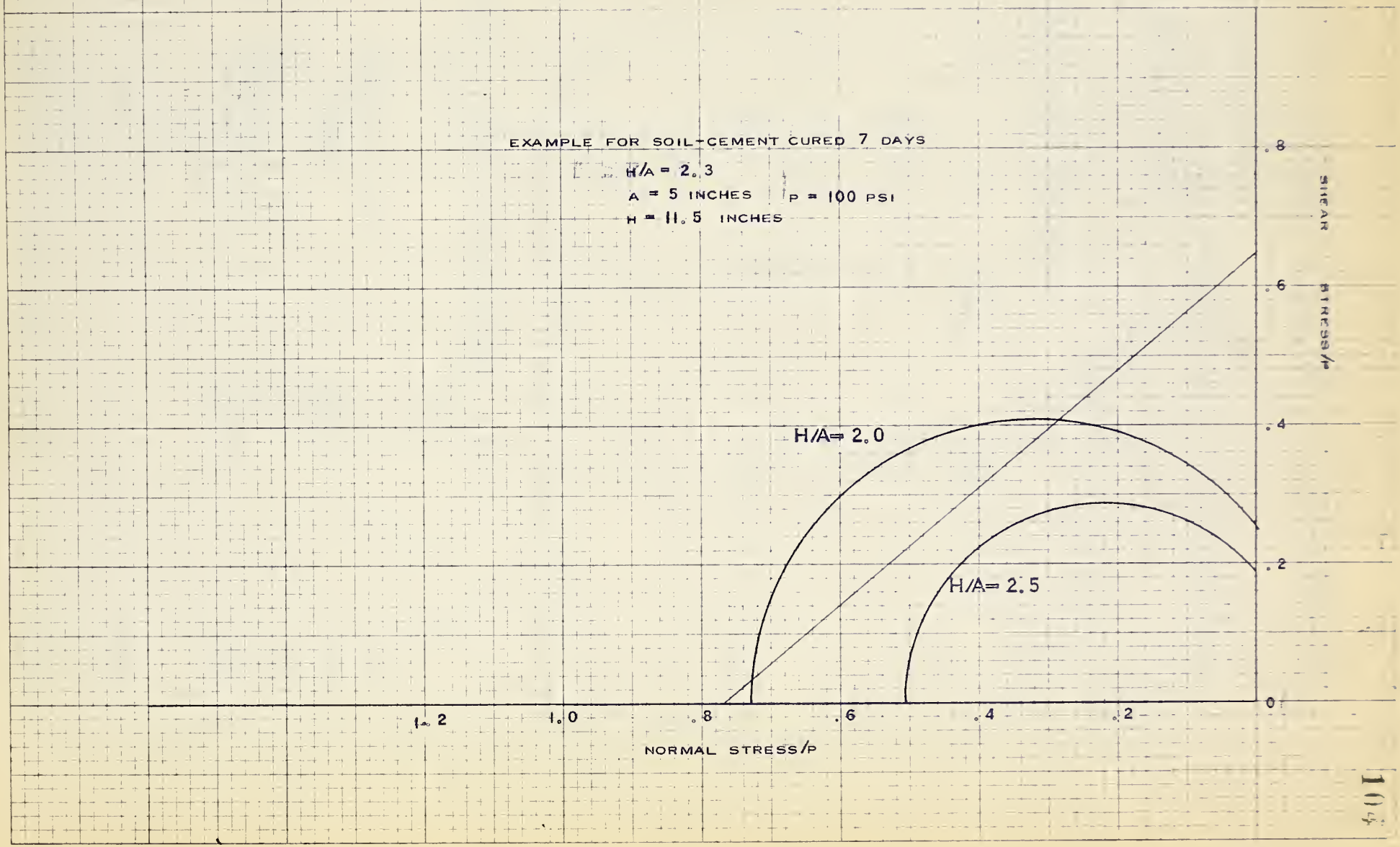


FIGURE 12

Sample Calculations for Analytical Solution and Confidence Limits for Mohr Envelopes (7)

For a sample computation, the 7 per cent MC3 cured sand-asphalt series will be used for illustration.

A summary of the minor (S_3) and major principal stresses for this series is listed below:

S_3(psi)	S_1(psi)
30	155
30	157
30	158
20	136
20	118
20	121
10	74
10	77
10	73
0	37
0	36
0	33

Computation of tangent to Mohr's circles for the data is presented in tabular form as follows:

	Item	S_3	S_1
	Sums	180	1175
	Sum of Products (S_1; S_3)	4200	23840
S_3	Product of Sums Divided by n	2700	17625
	Sum of Products (s_1, s_3)	1500	6215
	Sum of Products (S_1, S_3)		141185
S_1	Product of Sums Divided by n		115052
	Sum of Products (s_1, s_3)		26,133

s_1, s_3, x, y = stresses at failure in psi in terms of departures from their arithmetic means.

σ = standard deviation--psi

$$A^2 = \frac{\Sigma s_1^2}{\Sigma s_3^2} = \frac{26,133}{1,500} = 17.4$$

A = 4.16 (arbitrary symbol)

$$\tan \phi = \frac{A - 1}{2\sqrt{A}} = \frac{4.16 - 1}{2 \times 2.04} = 0.775$$

$$\underline{\phi = 37.8 \text{ degrees}}$$

$$\text{Cohesion} = \frac{\Sigma S_1 - A\Sigma S_3}{2n\sqrt{A}} = \frac{1175 - 4.16 \times 180}{2 \times 12 \times 2.04} = \underline{8.6 \text{ psi}}$$

$Y = 8.6 + 0.775X$

Y = shearing stress at failure in psi

X = normal stress at failure in psi

$$B^2 = \frac{\Sigma s_1 s_3}{\Sigma s_3^2} = \frac{6215}{1500} = 4.14$$

B = 2.03 (arbitrary symbol)

$$\sigma s_3^2 = \frac{\Sigma s_3^2}{n} = \frac{1500}{12} = 125$$

$$2\sigma Y'_{yx} = \sqrt{\frac{A - B^2}{n - 2}\left(2\sigma s_3^2 + \frac{(A + 1)^2 x^2}{A(A+B^2)}\right)}$$

$$= 0.50 + 0.00154x^2$$

$\sigma Y'_{yx}$ = standard error of the estimated shearing strength for the regression line of Y or X in psi.

M_x = arithmetic mean value of X in psi

$$M_x = \frac{1}{n(A + 1)}(\Sigma S_1 + A\Sigma S_3) = 31.2$$

$$x = X - M_x = X - 31.2$$

The confidence limits are tabulated below for any departure

x, from the mean of the normal stress.

x	X	Y	$(2\sigma Y')^2$	$2\sigma Y'$	Y + $2\sigma Y'$	Y - $2\sigma Y'$
-31.2	0	8.6	2.00	1.41	10.0	7.2
0	31.2	32.8	0.50	0.70	33.5	32.1
30	61.2	56.0	1.88	1.36	57.4	54.6
60	91.2	79.2	6.05	2.46	81.7	76.7
120	151.2	125.8	22.70	4.76	130.6	121.0

The calculations of the confidence limits are based on arbitrary chosen values for x, the mean of the normal stress.

Sample Calculations of Void Properties of Immersed Sand-Asphalt

For an example the 5 per cent MC3 immersed sand-asphalt series at a lateral pressure of 30 pounds per square inch will be selected.

Average weight before test of 3 specimens	=	1237 gms.
Average dry weight of 3 specimens	=	1104 gms.
Weight of water at testing	=	133 gms.
Assume specific gravity of residue asphalt	=	1.0
Residue asphalt with 5% MC3	=	3.75%
Weight of residue asphalt $= 1104 - \frac{1104}{103.75}$	=	38 gms.
Total volume of sample	=	640 c.c.
Volume of soil solids $= \frac{1066}{2.67}$	=	400 c.c.
Volume of voids = 640 - 400	=	240 C.C.
Voids filled with asphalt = 38/240	=	15.8%
Voids filled with water = 133/240	=	55.4%
Voids filled with air = 100 - 15.8 - 55.4	=	28.8%
Total voids as % of soil solids = 240/400	=	60 .0%
Total voids as % of total mix = 240/640	=	37.4%
Volume of sample	=	$.0226 ft^3$
Dry Density of Soil Solids $= \frac{1066}{454 \times .0226}$	=	104.1 lbs/ft^3
Dry Density of total mix (asphalt residue and soil solids) $= \frac{1104}{454 \quad .0226}$	=	107.8 lbs/ft^3

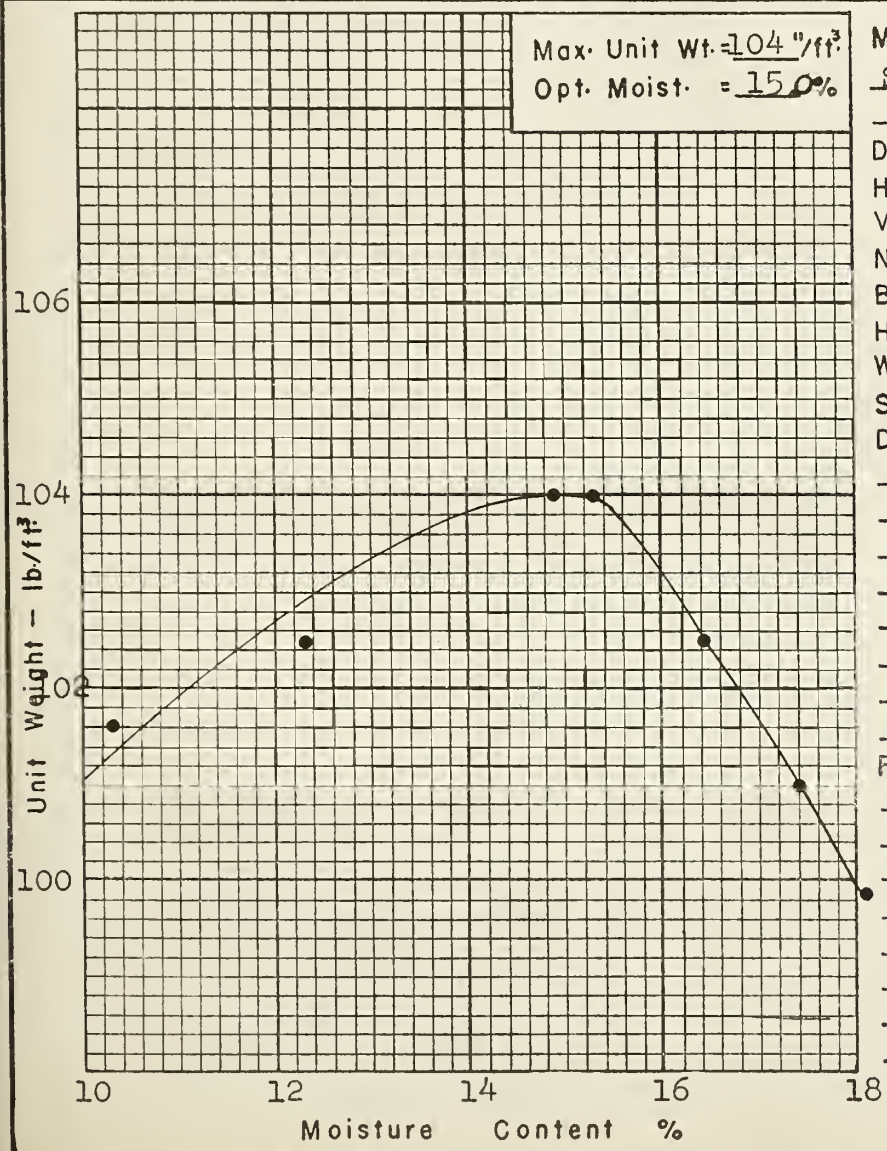
APPENDIX B

Detailed Data on Classification and Compaction of Sand

UNIVERSITY of ALBERTA
DEP'T. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
COMPACTION TEST

PROJECT	THESIS
SITE	McGinn Pit North
SAMPLE	Sand
LOCATION	Stony Plain, Alberta
HOLE	DEPTH
TECHNICIAN	D.G.P. DATE 11/28/61

	Trial Number	1	2	3	4	5	6	7
Unit Weight Determination	Mold No.							
	Wt. Sample Wet + Mold							
	Wt. Mold							
	Wt. Sample Wet Gms.	1696	1742	1812	1796	1780	1821	1812
	Volume Mold	1/30	1/30	1/30	1/30	1/30	1/30	1/30
	Wet Unit Weight lb./ft.^{3}	112.0	115.0	119.5	118.0	117.5	120.2	119.5
	Dry Unit Weight lb./ft.^{3}	101.6	102.5	104.0	101.0	99.4	104.0	102.5
Moisture Content Determination	Container No.							
	Wt. Sample Wet + Tare	692.02	738.35	663.19	713.00	697.83	608.60	561.80
	Wt. Sample Dry + Tare	648.92	661.93	589.10	623.32	597.40	559.14	497.00
	Wt. Water	43.10	76.42	74.09	89.68	100.43	49.46	64.80
	Tare Container	231.50	42.79	104.33	108.64	43.56	231.50	101.63
	Wt. Dry Soil	417.42	619.14	484.77	514.68	553.84	327.64	395.37
	Moisture Content	10.3	12.3	14.9	17.4	18.1	15.3	16.4



Method of Compaction Standard Proctor Compaction
Diam. Mold 4.00 inches
Height Mold 4.60 inches
Volume Mold 1/30 ft.^{3}
No. of Layers 3
Blows per Layer 25
Ht. of Free Fall 12 inches
Wt. of Tamper 5.5 lbs.
Shape of Tamping Face 0
Description of Sample

Remarks Standard Proctor Density to be duplicated in 2.80 inch diameter mold.

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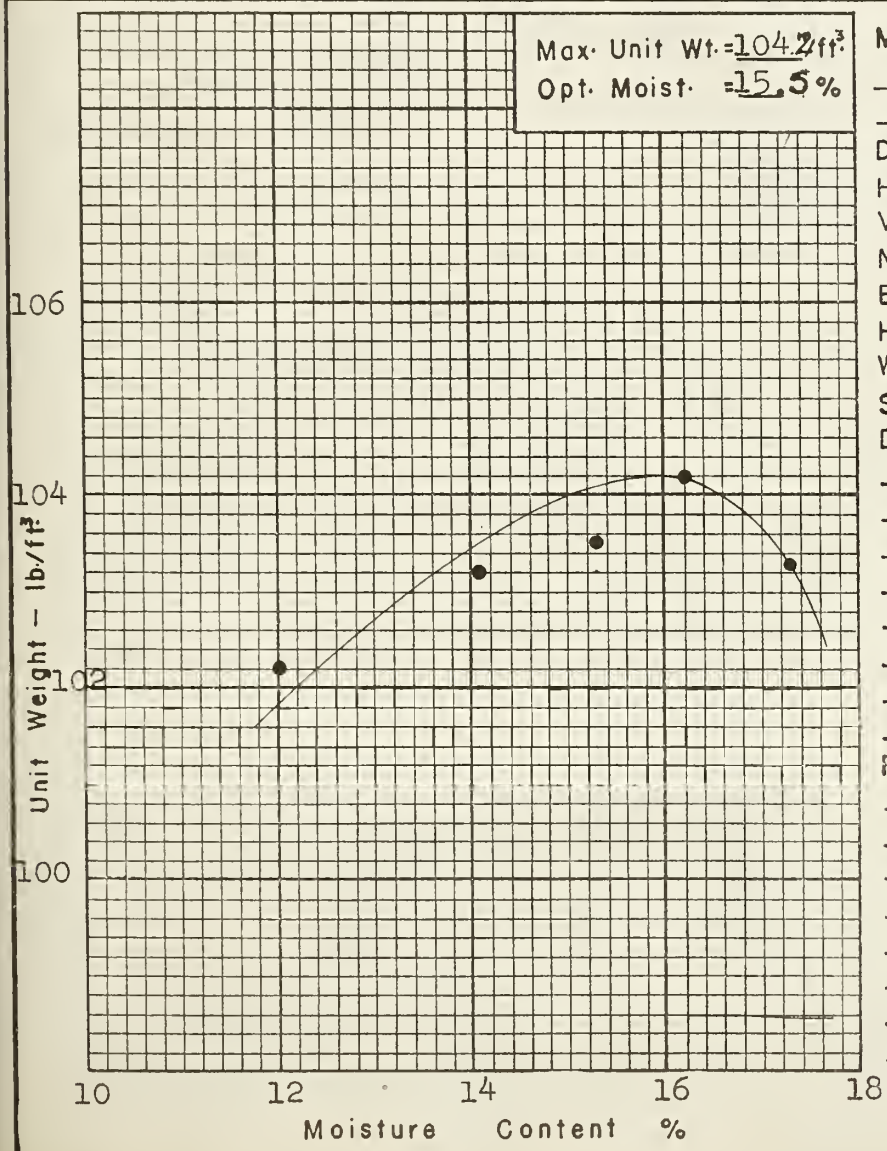
COMPACTION TEST

PROJECT THESIS
SITE McGinn Pit North
SAMPLE Sand
LOCATION Stony Plain, Alberta
HOLE DEPTH
TECHNICIAN D.G.P. DATE 1/5/62

	Trial Number Sample	R-12	R-14	R-15	R-16	R-16		
Unit Weight Determination	Mold No.							
	Wt. Sample Wet + Mold							
	Wt. Mold							
	Wt. Sample Wet Gms.	1178	1209	1225	1246	1246		
	Volume Mold							
	Wet Unit Weight lb./ft.^{3}	114.5	117.8	119.4	121.2	121.2		
	Dry Unit Weight lb./ft.^{3}	102.2	103.2	103.5	104.2	103.3		
Moisture Content Determination	Container No.	9	10	8	11	3A		
	Wt. Sample Wet + Tare	345.9	352.4	370.6	345.8	328.8		
	Wt. Sample Dry + Tare	320.2	322.6	336.7	312.2	296.2		
	Wt. Water	25.7	29.8	33.9	33.6	32.6		
	Tare Container	107.2	111.0	114.9	105.5	107.6		
	Wt. Dry Soil	213.0	211.6	221.8	206.7	188.6		
	Moisture Content	12.0	14.1	15.3	16.2	17.3		



Method of Compaction Drop Hammer with Solid Base Mold.
Diam. Mold 2.80 inches
Height Mold 6.50 inches
Volume Mold .00226 ft^3
No. of Layers 5
Blows per Layer 8,9,10,11,12
Ht. of Free Fall 18 inches
Wt. of Tamper 10.3 lbs.
Shape of Tamping Face o
Description of Sample

Remarks This Proctor Curve is that obtained by using the 2.80 inch diameter mold in the simulating of Proctor density in the standard mold

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SIEVE ANALYSIS

PROJECT THESIS
SITE McGinn Pit North
SAMPLE Sand
LOCATION Stony Plain, Alberta
HOLE DEPTH
TECHNICIAN D.G.P. DATE 12/18/61

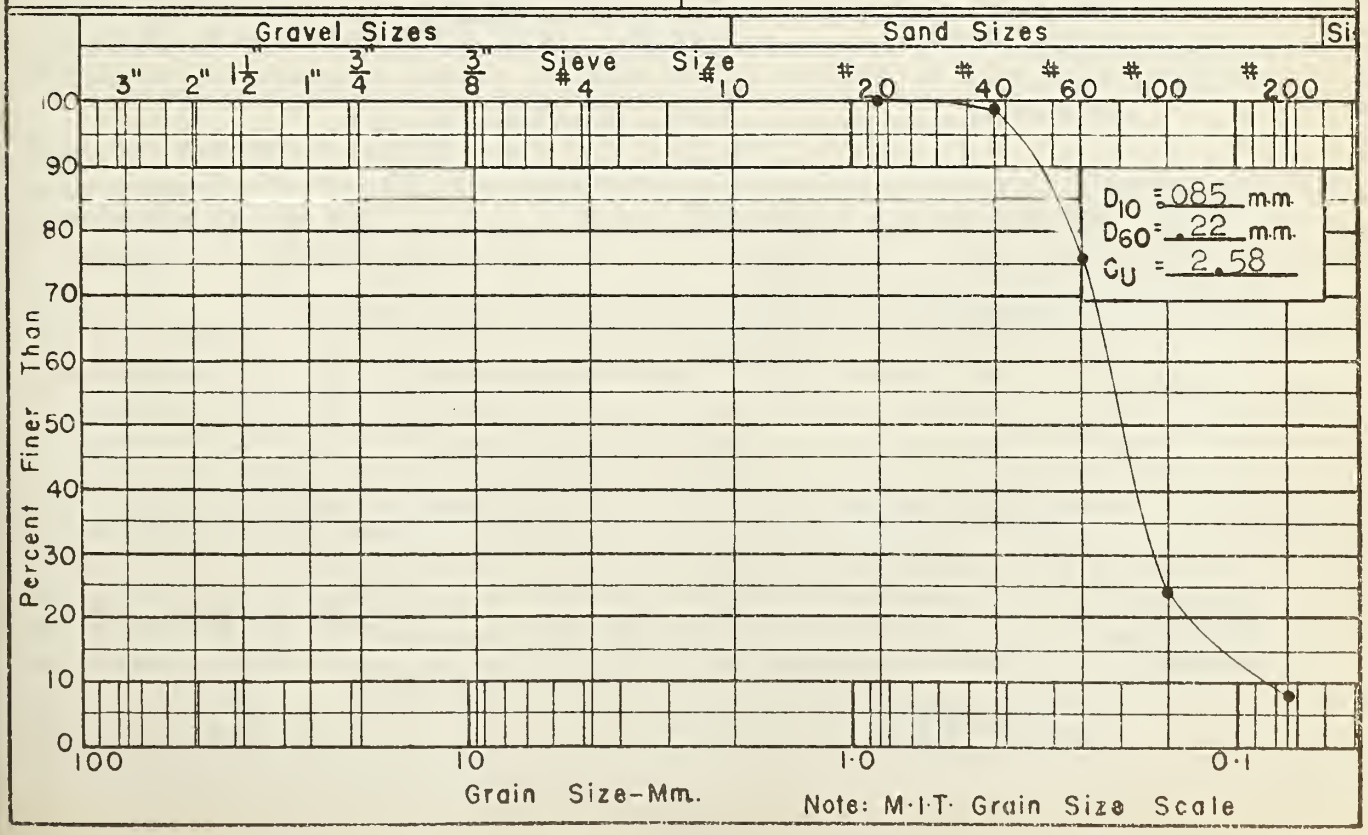
Total Dry Weight of Sample	Sieve No.	Size of Opening Inches	Size of Opening Mm.	Weight Retained gms.	Total Wt. Finer Than gms.	Percent Finer Than	% Finer Than Basis Orig. Sample
Initial Dry Weight Retained No. 4							
Tare No.							
Wt. Dry + Tare							
Tare		3/4	19·10				
Wt. Dry		3/8	9·52				
	4	·185	4·76				
Passing	4						
Initial Dry Weight Passing No. 4							
	10	·079	2·000				
Tare No.	20	·0331	·840	0.0	99.04	100%	
Wt. Dry + Tare	40	·0165	·420	0.7	98.34	99.3%	
Tare	60	·0097	·250	22.8	75.54	76.4%	
Wt. Dry 99.04	100	·0059	·149	51.0	24.54	24.7%	
	200	·0029	·074	16.3	8.24	8.1%	
Passing	200						

Description of Sample

Time of Sieving 15 Minutes

Method of Preparation Air-dried
Sample was washed through #100 & #200 sieves then ovendried and dry-sieved through the complete nest of sieves.

Remarks



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HYDROMETER TEST

PROJECT THESIS
SITE McGinn Pit North
SAMPLE Sand
LOCATION Stony Plain, Alberta
HOLE DEPTH
TECHNICIAN D.G.P. DATE 12/19/61

Date	Temp.	Time	Elapsed Time	R'_h	$R_h = R'_h + c_m$	D m.m.	$R_h + m_f - c_d$	W. %	W % Basis Orig. Sample	Remarks
12/19	25.4C	11:15	1/4min.	28.0	28.3	.0900	29.1	37.5	9.3	
"	"		1/2	19.0	19.3	.0750	20.1	25.5	6.3	
"	"		1	13.0	13.3	.0550	14.1	18.0	4.5	
"	"		2	9.0	9.3	.0420	10.1	12.8	3.2	
"	"		4	6.3	6.6	.0300	7.4	9.5	2.3	
"	"		8	4.5	4.8	.0210	5.6	7.1	1.8	
"	"		15	3.3	3.6	.0160	4.4	5.6	1.4	
"	"		30	2.4	2.7	.0110	3.5	4.5	1.1	
"	"		60	1.9	2.2	.0080	3.0	3.8	0.94	
"	"		120	1.8	2.1	.0057	2.9	3.7	0.91	
"	24.1C		240	1.5	1.8	.0041	2.6	3.3	0.81	
20/Dec	"	9:45	1350	0.6	0.9	.0017	1.7	2.2	0.55	

Hydrometer No.s 36431 and _____ Graduate No. _____

$W\% = \frac{100}{W_s} \cdot \frac{S_s}{S_s - 1}(R_h + m_f - c_d) = 1.27\ (R_h + m_f - c_d)$

Meniscus correction = c_m = 0.3@25.5C and _____ respectively

Dispersing agent used 6% Soln. of Calgon Amount 10 c.c.

Correction for change in density of liquid due to addition of dispersing agent = c_d

c_d = -0.3@25.5C and _____ respectively

Specific Gravity of Solids = G_s 2.67

Description of Sample _____

Method of Preparation Air-dried sample was sieved and the material retained on #200 was used in Hydrometer Analysis.

Remarks _____

Initial Moisture Content

Container No. _____
Wt. Sample Wet + Tare _____
Wt. Sample Dry + Tare _____
Wt. Water _____
Tare Container _____
Wt. of Dry Soil _____
Initial Moisture w % _____

Dry Weight of Sample

Container No. 9
Wt. Sample (~~Wet~~/Dry) + Tare 233.70
Tare 108.11
Wt. (~~Wet~~/Dry) Soil 125.61

Dry Weight from Initial Moisture = $\frac{100 \times \text{Wt. Wet Soil}}{100 + \text{Init. Moist.\%}}$ = _____

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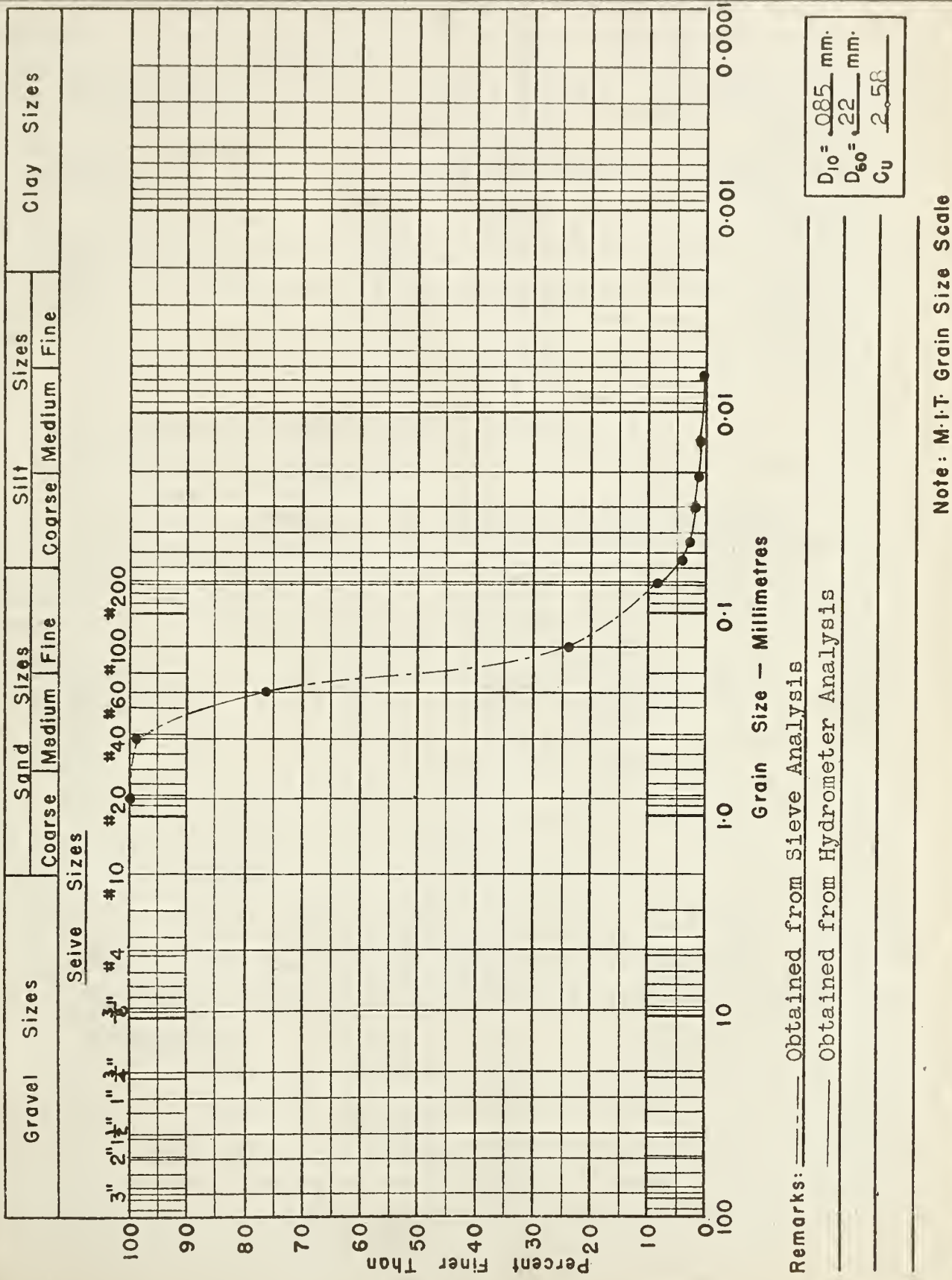
SOIL MECHANICS LABORATORY

GRAIN SIZE CURVE

PROJECT	THESIS
SITE	McGinn Pit North
SAMPLE	Sand
LOCATION	Stony Plain, Alberta
HOLE	DEPTH
TECHNICIAN	D.E.P. DATE 12/18/61



D_{10} = .085 mm.
D_{60} = .22 mm.
C_u = 2.58

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SPECIFIC GRAVITY

PROJECT THESIS
SITE Stony Plain, Alberta
SAMPLE Sand
LOCATION McGinn Pit North
HOLE DEPTH
TECHNICIAN D.G.P. DATE Dec.15/61

Sample No.	1	2
Flask No.	B1	B2
Method of Air Removal	Boil & Vacuum	Boil & Vacuum
W_{b+w+s}	794.08	789.18
Temperature T	26.8 Degrees C.	25.3 Degrees C.
W_{b+w}	701.40	695.98
Evaporating Dish No.	12	13
Wt. Sample Dry + Dish	255.70	256.70
Tare Dish	107.52	107.94
W_s	148.18	148.76
G_s	2.67	2.67

W_{b+w+s} = Weight of flask + water + sample at T°.
W_{b+w} = Weight of flask + water at T° (flask calibration curve).
W_s = Weight of dry soil
G_s = Specific gravity of soil particles $= \frac{W_s}{W_s + W_{b+w} - W_{b+w+s}}$

Determination of W_s from wet soil sample:

Sample No.			Sample No.		
Container No.			Container No.		
Wt. Sample Wet + Tare			Wt. Test Sample Wet + Tare		
Wt. Sample Dry + Tare			Tare Container		
Wt. Water			Wt. Test Sample Wet		
Tare Container			W_s		
Wt. of Dry Soil					
Moisture Content w %					

Description of Sample: Approximately 150 grams of air-dry sand was used in the two specific gravity determinations. Deairing was accomplished by boiling and deairing with a vacuum for 15 minutes.

Remarks:

APPENDIX C

Sample Data Sheet

UNIVERSITY OF ALBERTA
DEPT. OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
SAND-ASPHALT STABILIZATION
TRI-AXIAL & COMPACTION DATA

Project THESIS
Site McGinn Pit Stony Plain, Alta
Sample Sand
Technician D.G.P.
Date January 11, 1962

Specimen Number	58	60	61		
Asphalt Content	5%	5%	5%		
Lateral Pressure--psi - σ_{III}	10	10	10		
Length--in.	6.45	6.42	6.44		
Area--in^2	6.05	6.09	6.06		
Volume--ft^3	.0226	.0226	.0226		
Wet Wt. After Compaction	1261	1259	1258		
Wet Unit Weight	123.0	122.9	122.8		
Dry Unit Weight	108.8	108.8	108.7		
G_s = 2.67 Volume Soil Solids					
Void Ratio					
Curing Time--hrs.	7 Day Cure at 100° F.				
Wt. Sample Before Testing	1124	1121	1122		
Dry Weight of Sample	1115	1114	1113		
Wt. of Volatiles After Comp.	146	145	145		
Volatile Content at Comp.	13.5	13.5	13.5		
Wt. of Volatiles at Testing	9	7	9		
Volatile Content at Testing	0.8	0.6	0.8		
Strain Rate .01 in./min.					

Prov. Ring Div.	Load	Strain Read.	Strain	Corr. Area	σ_I	$\sigma_I - \sigma_{III}$
Specimen #58						
45	42	65	.001	6.05	17.0	7.0
90	85	130	.002	6.06	24.0	14.0
128	122	195	.003	6.07	30.1	20.1
185	177	260	.004	6.07	39.2	29.2
237	227	325	.005	6.08	47.3	37.3
290	278	390	.006	6.08	55.7	45.7
336	322	455	.007	6.09	62.8	52.8
379	364	520	.008	6.10	69.7	59.7
414	398	585	.009	6.11	75.2	65.2
442	424	650	.010	6.11	79.5	69.5
481	472	780	.012	6.12	87.0	77.0
502	482	910	.014	6.14	88.6	78.6
499	480	1040	.016	6.15	88.2	78.2
496	477	1170	.018	6.16	87.6	77.6
484	465	1300	.020	6.17	85.3	75.3
Specimen #60						
202	193	130	.002	6.11	41.6	31.6
277	265	195	.003	6.11	53.4	43.4
349	335	260	.004	6.12	64.8	54.8
412	396	325	.005	6.12	74.6	64.6
465	446	390	.006	6.13	82.8	72.8

Prov. Ring Div	Load	Strain Read.	Strain	Corr. Area	σ_I	$\sigma_I - \sigma_{III}$
503	483	455	.007	6.13	88.6	78.6
530	509	520	.008	6.14	93.0	83.0
546	525	585	.009	6.14	95.5	85.5
556	538	650	.010	6.15	97.5	87.5
560	542	715	.011	6.15	98.3	88.3
561	543	780	.012	6.16	98.2	88.2
559	541	845	.013	6.17	97.6	87.6
536	520	1040	.016	6.19	93.0	83.0
Specimen #61						
187	179	65	.001	6.06	39.6	29.6
300	287	130	.002	6.07	57.3	47.3
372	358	195	.003	6.08	68.8	58.8
430	414	260	.004	6.08	78.1	68.1
470	452	325	.005	6.09	84.2	74.2
496	476	390	.006	6.09	88.2	78.2
513	494	455	.007	6.10	91.0	81.0
522	503	520	.008	6.10	92.6	82.6
523	502	585	.009	6.11	92.5	82.5
517	497	650	.010	6.13	91.3	81.3
505	485	780	.012	6.14	89.0	79.0

REMARKS #58 - Bulging Failure & Shear
#60 - Shear Failure
#61 - Shear Failure

AVERAGE S_1 = 93.2
FOR S_3 = 10 psi